Report No. 3

EVALUATION OF PRESENT STATE OF KNOWLEDGE OF FACTORS AFFECTING TIDAL HYDRAULICS AND RELATED PHENOMENA

C. F. Wicker, Editor

May 1965

Note

This report is a completely revised edition of Report No. 1 of the Committee, same title, dated February 1950

Committee on Tidal Hydraulics

CORPS OF ENGINEERS, U. S. ARMY

REPORTS OF COMMITTEE ON TIDAL HYDRAULICS

Report No.	Title	Date
1	Evaluation of Present State of Knowledge of Factors Affecting Tidal Hydraulics and Related Phenomena	Feb. 1950
2	Bibliography on Tidal Hydraulics	Feb. 1954
	Supplement No. 1, Material Compiled Through May 1955	June 1955
	Supplement No. 2, Material Compiled from May 1955 to May 1957	May 1957
	Supplement No. 3, Material Compiled from May 1957 to May 1959	May 1959
	Supplement No. 4, Material Compiled from May 1959 to May 1965	May 1965

Technical Bulletin No.	Title	Date
1	Sediment Discharge Measurements in Tidal Waterways	May 1954
2	Fresh Water-Salt Water Density Currents, A Major Cause of Siltation in Estuaries	April 1957
3	Tidal Flow in Entrances	Jan. 1960
4	Soil As A Factor in Shoaling Processes, A Literature Review	June 1960
5	One-Dimensional Analysis of Salinity Intrusion in Estuaries	June 1961
6	Typical Major Tidal Hydraulic Problems in United States and Research Sponsored by the Corps of Engineers Committee on Tidal Hydraulics	June 1963
7	A Study of Rheologic Properties of Estuarial Sediments	Sept. 1963
8	Channel Depth as a Factor in Estuarine Sedimentation	Mar. 1965

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CORPS OF ENGINEERS, U. S. ARMY

ARMY-MRC VICKSBURG, MISS

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COMMITTEE ON TIDAL HYDRAULICS

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FOREWORD

The U. S. Army Engineer Committee on Tidal Hydraulics was established by the Chief of Engineers with the following objectives:

<u>a</u>. To maintain a continuing evaluation of the state of knowledge required for the improvement and maintenance of tidal waterways.

<u>b</u>. To recommend studies, investigations, and research designed to provide the knowledge necessary to arrive at adequate solutions for the engineering problems associated with tidal waterways.

<u>c</u>. To exercise advisory technical supervision of assigned programs.

<u>d</u>. To publish and disseminate pertinent information.

<u>e</u>. To render such consulting service on specific problems in tidal waterways as may be requested by various organizations of the Corps of Engineers.

In February 1950, the Committee published its Report No. 1, <u>Evaluation of Present State of Knowledge of Factors Affecting Tidal Hydraulics and Related Phenomena</u>, as its initial accomplishment under the first of the objectives listed above. It revealed the existence of very large areas of uncertainty or ignorance which make analyses of the causes of tidal waterway problems difficult; these difficulties in turn cause the conclusions drawn to be of doubtful accuracy. In accordance with objective <u>b</u>, above, the Committee has recommended programs designed to provide knowledge in certain of the areas where the deficiencies were greatest, or were of the most serious import, and some of them have been authorized and are either completed or are well advanced. Enough has been learned from these programs to warrant publication of a completely revised "Evaluation" report. The revision bears the same title as Report No. 1, but to avoid confusion, is issued as Report No. 3.

This report is not intended to be a textbook or a manual on tidal hydraulics. Rather it is intended to acquaint the reader with the various factors which should be considered in solving problems involving tidal hydraulics and to evaluate the present state of knowledge of these factors. The references given at the end of each chapter will supply definitive information on the subjects presented in the various chapters.

It will be noted that the revised report is not the work of one author, thus following the precedent of the original report. The authors of the individual chapters are the members of and consultants to the Committee, and the assignments were based on special interest of the author or authors in the subject.

ACKNOWLEDGMENTS

This report is a joint effort of the members of the Committee on Tidal Hydraulics of the U. S. Army Corps of Engineers and consultants under contract to the Committee. Specific credit for each chapter is given in the Contents and at the beginning of each chapter. It is to be recognized, however, that the original drafts of each chapter were critically reviewed by the other committee members and two or more of the consultants. Thus, each chapter reflects to a degree the experiences and thoughts of the committee members and the consultants.

The committee members who participated in the preparation of the report were:

Joseph M. Caldwell	Coastal Engineering Research Center*
Jacob H. Douma	Office, Chief of Engineers
Richard O. Eaton**	Coastal Engineering Research Center
James R. Johnston	North Atlantic Division
Clement P. Lindner	South Atlantic Division
John B. Lockett	North Pacific Division
John B. McAleer†	New England Division
John C. Marcroft††	South Pacific Division
Edward A. Schultz	San Francisco District
Henry B. Simmons	Waterways Experiment Station
Joseph B. Tiffany	Waterways Experiment Station
Clarence F. Wicker‡	Philadelphia District

Consultants who participated in the preparation of the various chapters were:

Arthur T. Ippen	Massachusetts Institute of Technology
Garbis H. Keulegan‡‡	National Bureau of Standards
Donald W. Pritchard	Johns Hopkins University
Clarence F. Wicker	Private Consultant

New members who joined the Committee subsequent to the initial drafting of the report, but who assisted in the review of the report before publication, were:

P. Alfred Becnel, Jr.	New Orleans District
Albert B. Davis, Jr.	Galveston District
Thorndike Saville, Jr.	Coastal Engineering Research Center

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* The Coastal Engineering Research Center is successor to the Beach Erosion Board, which was dissolved in November 1963.

** Mr. Eaton retired in December 1963.

† Mr. McAleer transferred to Office, Chief of Engineers, in December 1964.

†† Mr. Marcroft retired in July 1963.

‡ Mr. Wicker retired in January 1962 and was immediately put under contract as a consultant to the Committee. He was largely responsible in this latter capacity for the final editing of the ten chapters of this report and reconciling the various comments resulting from the reviews by members and consultants.

‡‡ Dr. Keulegan retired in 1960 and is now employed part time at the Waterways Experiment Station.

CONTENTS

CHAPTER I

CONSIDERATIONS IN THE IMPROVEMENT OF TIDAL WATERWAYS

by

J. H. Douma and C. F. Wicker

Introduction

I-1. It is frequently discovered that a tidal waterway which is strategically located with respect to centers of population or sources of raw materials, or which serves as a thoroughfare for vessels enroute between such terminals, is not in its existing state adequate to fulfill the requirements of modern shipping. The channel may be shallow or tortuous, the current therein may be very strong, or it may be exposed to storms. Vessels may be delayed by these conditions, or they may even find it impossible to use the waterway. The economic losses thus caused generate demands that the unsatisfactory conditions be corrected. Sometimes, as in the case of the situations at Suez, Jutland, and Cape Cod, the creation of a tidal waterway where none previously existed was desired in order to shorten sailing distances or to eliminate the need for navigating a particularly dangerous natural passage.

I-2. It is unwise to undertake such improvements without a thorough understanding of the possible consequences. A tidal waterway has a subtle personality that may mislead the unwary into undertaking difficult and expensive works that do not fulfill their purpose, or cause undesirable effects. Usually, the poor performance is due to the interplay of forces and factors either unrecognized or underrated by the engineer. The more deeply he delves into these forces and factors, the more keen is his realization that a waterway affected by tidal forces is a very complicated problem area.

I-3. The branch of knowledge applicable to such studies has become known as "tidal hydraulics." There is reason to regret the adoption of the term to include all tidal waterway engineering, as many are prone to consider its scope to be limited to the rise and fall of the water surface in consonance with the movements of heavenly bodies that generate the forces, and to the currents that are caused by the alternately rising and falling tide. It is emphasized that the term "tidal hydraulics" has come to be understood as including, in addition to the purely hydrodynamic considerations of such tidal waterways as inlets, estuaries, maritime straits, and canals, the following: channel dimensions and alignment; shoaling, including consideration of sources of the sediment, manner of transport, and cause of deposition; training works and dredging procedure (but not dredge design); jetty and breakwater layout; the salinity of the water, including phenomena associated therewith; and the dispersal and flushing of pollutants.

I-4. One of the most common disappointments experienced by an engineer charged with responsibility for the design of a deeper channel in a tidal waterway is the discovery that the new channel shoals very rapidly. It is possible that a different layout would have resulted in a much lower rate of shoaling, or perhaps the simultaneous construction of training works would have prevented the shoaling. Possibly the channel project would have been discarded before work was undertaken if there had been a realization that the shoaling rate would be excessive. An understanding based on present-day knowledge, even though deficient, of the factors involved in the transportation and deposition of sediments in tidal waterways and the effects of the new channel itself on these factors would go far in sparing the engineer the shock that will be his when he discovers that the new channel cannot be maintained economically.

I-5. He may be embarrassed to find that the new channel has so drastically altered the regimen of the waterway that the tide rises higher and falls lower than was the case before the improvement. Higher tides may injure shoreline property and cause claims for damages; lower tides may result in channel depths less than were designed, thus necessitating redredging at a cost much exceeding that initially expected. Salinity intrusions may be worsened, requiring sources of water supply to be abandoned. Training works proposed to cure shoaling problems may be ineffective. Jetties at entrances to tidal waterways may fail to provide the sheltered access desired, may fail (after the passage of a few years) to facilitate the maintenance of channels over outer bars, and may cause serious erosion of beaches downdrift of the entrance. Breakwaters may fail to provide the desired protection from storm waves, and may so interfere with shoreline processes as to cause erosion. The present state of the knowledge is sufficient to minimize the chance of occurrence of such difficulties, even though there are areas requiring additional investigation.

Factors Affecting Tidal Waterways

I-6. Tidal phenomena occurring in any waterway seldom result from a single cause, but are a more or less complex interaction of a number of factors. Thus, if a change in the regimen of a waterway is desired in order to effect an improvement, the change in each contributing factor and in the resulting interaction must be determined. The principal factors to be taken into consideration are: tides, tidal currents, freshwater discharge, volume of sediment, characteristics of beds and banks, wave action, littoral processes, salinity intrusion, and dispersal and flushing of pollutants.

The tide

I-7. The tide obviously is the generating force that either singly, or in interplay with other forces, causes the rise and fall of the water surface and the resultant currents. It is in itself a phenomenon that defied analysis by generations of devoted scholars. Only relatively recently has it been possible for them to explain why the tide at Passamaquoddy, Maine, rises and falls about 18 ft twice in a little over a day, while at Galveston, Texas, the mean fluctuation is approximately 1 ft and sometimes one of the daily rises and falls fails to occur. On the east coast of the United States, the tide repeats itself fairly closely twice daily (24 hr 50 min) while on the west coast, there is a considerable difference between the morning and afternoon tides.

I-8. These basic differences in the characteristics of the tides in the oceans adjacent to the shore make it dangerous to extend conclusions drawn as a result of a careful study of a tidal waterway on the east coast to a problem concerning a waterway on the gulf coast or the west coast. In fact, the tide is so greatly affected by the geometry of a waterway as it progresses to the upper limits that it is almost as dangerous to consider that the tidal aspects of problems on two west coast (or two east coast) waterways may be similar, without close scrutiny.

I-9. It is almost unnecessary to emphasize the importance of this factor in tidal waterway design and in connection with studies of problems. The tide is as significant in the life of the waterway as the pulse of blood through the arteries is to human life. It sets the whole tone of the waterway, and it is certainly a consideration in determining the available depth for navigation as well as in ship's cargo loading and unloading operations.

Tidal currents

I-10. The tidal currents are generated by the vertical rise and fall of the water surface and modified by the geometry of the waterway, freshwater discharges into the waterway, and salinity. In the case of straits and tidal canals connecting two bodies with independent tides (as for example Cape Cod Canal or Suez Canal), the regimen of currents is a complicated resultant of the forces generated by the two tides. As the tides vary somewhat from day to day due to astronomic considerations as well as meteorological variations, there is a very wide gamut of combinations that will enter into the resultant current.

I-11. The currents obviously are the means for the transport of sediments. Local variations in their strength will cause scour or deposition, depending upon the sediment itself. The general variation in strength in consonance with the pulse of the tide (which causes the flow to vary from zero velocity to a maximum in one direction, then back to zero, reverse direction, increase in velocity to a maximum, then return to zero only to repeat the process) causes alternate deposition and scour of sediments.

Tidal theories

I-12. Tidal theories are available to explain adequately the rise and fall of the tide in the ocean, but the engineer concerned with a problem waterway need not seek to rationalize the basic tide data at the entrances. Of course, he needs to know the characteristics of the basic tide or tides, and then he must have an understanding of the factors that will modify the tide as it ascends or traverses the waterway. Among these are included the geometry of the waterway and the upland freshwater discharge. There have been many methods proposed for correlation of cause and effect, and for predicting the effect on the tide of a change in one or more of the factors along the course of the waterway. Among these may be listed the reflected wave technique employed by Colonel Earl I. Brown, CE, the methods proposed by Brig. General G. B. Pillsbury, CE, and the so-called method of characteristics employed by Netherlands government engineers. These are discussed in Chapter II.

Freshwater discharges

I-13. The freshwater discharges into the tidal waterway have profound effects on the regimen. They affect the basic tide independently of the effects of geometry, greatly modify the resultant current by lengthening the ebb and shortening the flood, transport upland sediment to the tidal waterway, and interact with salinity intrusion forces to produce a complicated modification of the distribution of currents in the vertical. Additionally, the inflow of fresh water is the means by which a tidal waterway purges itself of pollutants introduced by man, and it is the factor that, according to its strength (volume of inflow), prevents salinity intrusions beyond various points.

I-14. It should be obvious that failure to consider the effects of freshwater inflows almost automatically assures that a plan of improvement will be deficient, particularly if the freshwater inflow is an appreciable part of the tidal ebb flow. For example, as a result of reservoir construction and operation, serious shoaling may be created in an estuary or the severity of existing shoaling may be increased or reduced, depending on reservoir operation. Reservoir operation usually reduces the length of channel over which major shoaling occurs; thus, the resulting relation between locations of major shoal areas and existing and potential spoil disposal areas may be improved or worsened.

Volume of sediment

I-15. The volume of sediment brought down from the watershed beyond the influence of tide must be either removed from the waterway by dredging, or purged out to sea by the currents if the waterway is to remain in existence. It is not an exaggeration to state that a tidal waterway can die as a result of a continuing supply of upland sediment without removal of a corresponding volume, for there are historic references to such occurrences. In the case of waterways containing improved channels, the volume of inflow of sediment may be close to the volume of required maintenance dredging (see Chapter III).

Characteristics of bed and banks

I-16. The characteristics of the bed and banks of the waterway are highly significant factors. If they are composed of rock, or if large boulders are plentiful on the bed, it will be expensive to provide a deeper or wider channel, and in many cases the improvement may be uneconomic. Sandy or silty bottoms and banks, while easily dredged, are also easily scoured by swift currents, thus adding material to the burden to be transported and perhaps deposited in a locality requiring frequent dredging. One of the early investigations that should be undertaken as part of any study is an investigation of the characteristics of the bed and banks of the waterway wherever it is found that the currents are of greater than usual strength by reason of some peculiarity of the regimen, or where it may be anticipated that acceleration of currents will result from a proposed improvement of the channel.

Wave action

I-17. Wave action may be worthy of consideration in portions of the waterway where the banks are composed of unconsolidated materials. Natural wave action will be of significant force where the waterway is wide, and wave action generated by passing vessels will be important in narrow waterways. It may be that the volume of sediment thus brought into transport, also the value of land destroyed by the erosion, may be too small to be of significance. On the other hand, the situation may be such (as in the case of very high banks) that the result of bank caving due to wave action cannot be accepted.

I-18. At the entrance to the waterway, ocean waves will affect the configuration of bars which generally project across and into the opening. They also make navigation difficult and at times impossible. Cognizance of their directions of approach, especially when they are of considerable magnitude, should be taken in laying out bar channels. The other characteristics of the waves (height, period, and length) are needed for the design of any jetties or breakwaters considered necessary at the entrance.

Littoral processes

I-19. Littoral processes is the generic term applied to the methods employed by nature for molding and remolding shorelines. These processes are significant to investigations of tidal waterway matters primarily because their product is a given quantity of detritus moving along the shore. This represents the resultant of the erosion and accretion occurring along a section of shoreline having a fairly uniform exposure to waves, and which is not divided by inlets, deeps, or headlands such that passage of the detritus is impossible. The general term for such material is littoral drift.

I-20. Littoral movement of material may occur along the shoreline of the waterway where there is wave action of such character as to be powerful enough to place in suspension littoral material, and so directed as to cause a movement along the shore. It is desirable to investigate the littoral processes of the shoreline of the waterway itself to an extent sufficient to establish whether the eroded material ultimately moves into the navigable channels of the waterway.

I-21. Of greater significance is the effect of the littoral processes in depositing material offshore adjacent to the entrance to the tidal waterway. This is the material that produces the bar formations usually found at entrances; these are especially feared by navigators because their locations are not fixed and the best sailing course varies accordingly. Because of exposure, the channel threading through a bar formation is very difficult and expensive to maintain (see Chapter VI).

Salinity intrusions

I-22. Salinity intrusions are caused by the difference in densities of the salt water of the ocean and fresh water acting singly or in concert with the flood and ebb flows of the waterway. The dissolved solids in the ocean water, consisting mostly of sodium chloride but including a number of other salts, contaminate the water of the tidal body and thus impair its value as a source of domestic and even industrial water supply. They also cause curious changes in the hydraulics of the waterway. As a result, it may be found that sediments in transport in the waterway are deposited to form a repetitious shoal at a location not anticipated from the geometry (see Chapter V). In addition to these effects, it is known that certain dissolved salts will cause flocculation of certain suspended sediments, causing deposition.

I-23. The factors that must be recognized in investigating existing or prospective salinity intrusions are the tidal characteristics of the waterway, its geometry, and the runoff of fresh water entering the tidewater portion of the waterway. Major changes in channel dimensions may affect the extent of salinity intrusions by increasing the vertical density gradients, or as a result of changes in the tidal regimen. Major alteration in the runoff of fresh water, as for example by storage or diversion from or to the waterway, may significantly alter the salinity regimen with resultant modification of the shoaling pattern (and perhaps the volume) (see Chapters IV and V).

Dispersal and flushing of pollutants

I-24. Dispersal and flushing of pollutants have somewhat lately gained recognition as being factors to be considered in improving tidal waterways. It is conceivable that there is an existing balance between the total load of pollutants and the regimen of the waterway such that objectionable conditions do not exist. Alteration of the waterway or modification of the freshwater discharges into the waterway may cause increased or reduced concentrations of pollutants.

I-25. Obviously, the introduction of pollutants is undesirable. However, it may not be possible legally to require that the practice be discontinued, and it must be kept in mind that accidental spillages of contaminants may occur. It is quite important that consideration be given to the characteristics of the waterway in relation to its ability to disperse and flush away pollutants (see Chapter VIII).

Planning Tidal Waterway Improvements

I-26. As a basis for functional planning and design of a tidal waterway improvement, the factors previously discussed should be considered to establish the extent to which each is likely to influence the plan of improvement and its maintenance. It may be necessary to include hydrographic surveys, current observations, and sediment sampling in the investigations. Analysis of the factors involved, consideration of the effects beyond the problem area in terms of public interest, and determination of possible construction methods and availability of materials usually will suggest more than one method of improvement which will satisfy navigation requirements. In selecting the best method, preliminary plans should be prepared for all the methods believed capable of producing the desired end result and compared economically.

I-27. In addition to a thorough consideration of the existing physical conditions of and in the waterway, a study should be made of the changes which have occurred over the years, particularly in the vicinity of the entrance. Such a study would include comparisons of earlier maps, dating as far back as they are available, and discussion with persons having longtime informed acquaintance with the waterway. If any progressive or cyclic changes are disclosed by this study, the causative explanation should be sought. The planner should give full consideration to any seemingly "natural" changes that might have transpired, because of their possible effect on his project.

Hydrographic surveys

I-28. Hydrographic surveys constitute a prime requirement for intelligent study of a waterway improvement. They are an obvious necessity for use in studies concerning the location of a desired improvement. They are of equal importance for studies of the hydraulics of the waterway with a view to determining the effects of the improvement on the regimen, and for estimating the maintenance requirements of the proposed channel.

I-29. The data assembled during the survey should include, in addition to soundings, information on the tidal establishment to permit determination of datum planes for the survey, shoreline configuration to the head of tide (to include islands, tidal tributaries, low-lying contiguous areas subject to inundation by ordinary tide), and the character of the bottom down to a depth as great as that of the contemplated channel. It is always helpful to have such data from earlier surveys in order to establish trends for change.

I-30. A survey of the scope described generally in the preceding paragraph is an expensive operation. It grows more expensive as greater detail is demanded, but this fact is frequently overlooked or considered lightly. The criterion that should be employed in deciding matters in this area consists of providing adequate detail for the purpose intended but no overgenerosity. The engineer should cross-examine himself along the lines of exactly what purpose will the data serve, and whether data of lesser detail will serve as well. As an example, if survey crosslines 500 ft apart, plotted to a scale of 1 in. = 500 ft, will provide an estimate of the number of cubic yards to be removed in providing a channel within plus or minus one percent, there is surely no reason to prescribe a survey involving lines 100 ft apart plotted to a scale of 1 in. = 100 ft.

Current observations

I-31. Current observations, to include velocity and direction, should be made in the detail and extent necessary to determine whether conditions exist, or probably will exist following the improvement, such that navigation is made difficult or hazardous. The same information is required for use in the analysis of shoaling problems.

I-32. A program of current velocity and direction observations is difficult to execute properly, and it certainly is expensive. For these reasons, it is obviously necessary that very careful consideration be given to the exact purposes to be served by the data, and to the available capabilities for detailed planning and accomplishment of the observations. It is likely that lack of thought as to these matters will produce results that do not serve the purposes intended, or may even lead to erroneous conclusions.

Sediment sampling

I-33. Sediment sampling is undertaken as part of an investigation of a prospective shoaling problem. The work includes collection of samples of the bed of the stream in localities where scour is in progress, or may be expected following completion of the improvement, and where shoaling occurs and is likely to continue under the conditions that will exist with the improvement. One purpose of this work is to trace material scoured from the bed and banks of the waterway to a shoal if there is some distinctive mineral present at both locations to serve as a marker or identification. Generally, this does not occur in nature, and the principal purpose of this phase of a sediment sampling program is to provide general information on the material in the shoals.

I-34. Another feature of work that generally is undertaken is the collection of samples of the material in transport. The purposes of such information should be limited to the obtaining of a "broad-brush" understanding of the intensities of movement, its phasing with respect to currents, and its locations in the cross section. It may be found, for example, that the transport occurs close to the bottom exclusively. It follows then that the current regimen in close proximity to the bottom is the only agent involved in the movement, and thus should be understood in detail.

Navigation requirements

I-35. Navigation requirements dictate channel dimensions and layout. Required depth is premised upon the static draft of the largest vessels that will navigate the channel, their squat (sinkage), the least density of the water, and an empirical factor, generally taken as about 2 ft, to be added to the sum of all of the other factors enumerated above to provide a clearance between the keel and the bottom. In exposed channels, an additional allowance is necessary to provide for the pitching and rolling due to wave action.

I-36. The required width of the channel is determined on the bases of the beam of the largest vessel to use the channel, the beam of the largest vessel that it must pass if two-way traffic is permitted, and the desirable degree of ship control when ships pass each other to navigate bends. As closely passing ships interfere considerably with the flow of water about each other, which tends to change the ships' courses, the channel width must be sufficient to prevent undesirable loss of ship control. In traversing a bend, the ship is slightly off center, which may increase the difficulty of maneuvering sufficiently to require greater channel width, particularly in sharp bends.

I-37. The alignment of the channel is a consideration of perhaps greater significance to safe navigation than alignment of a highway is to safe passages of automobiles. Vessels do not turn to change direction; they literally slide or slither into the desired direction. While they are doing this, they are subjected to forces that have no counterparts in highway traffic. Currents rarely are aligned perfectly along sailing courses. Pressures which are generated as the vessel approaches the sides of the channel in making its turn seriously affect the steering and may cause it to depart suddenly from course (see Chapter X).

Interior navigation channels

I-38. Interior navigation channels may either modify to a large degree the regimen of a waterway, as in the case of a tidal river, or make comparatively little change in existing hydraulic conditions, as in the case of a large estuary. The basic considerations are selection of channel location and cross-sectional areas necessary to fulfill navigation requirements and assure project depth without excessive maintenance dredging.

I-39. Navigation channels should be made as straight as practical, with easy curvature, and should be aligned to conform as nearly as possible with the direction of predominant currents. When flood and ebb currents in a given reach follow different traces, a compromise alignment may result in the best current conditions but an alignment along the ebb trace usually produces the best channel depths.

I-40. If the channel is enlarged, sediment may deposit in the enlarged section and contraction works may be needed to prevent such deposits. Channel deepening modifies density current effects and thus affects shoaling in the saline region of the estuary, even though cross-sectional areas may be held constant by construction of contraction works. If practicable, the channel should be located to avoid areas which are most conducive to shoaling.

I-41. In planning a navigation channel for a relatively narrow waterway, hydrographic survey charts should be examined to determine the widths between banks at cross sections where project depth exists. A smooth curve drawn through these widths plotted against distance along the waterway will furnish a guide in estimating the width corresponding to project depth which will experience minimized shoaling.

I-42. The use of such a design procedure may eliminate the need for more elaborate hydraulic investigations in some waterways, as the channel produced by the river over a long period integrates the effects of all factors involved. Thus, the engineer's problem is reduced to determining the practical and economic feasibility of reproducing self-maintaining sections of desired depth by constructing control works and whether this method is preferable to maintaining the channel by dredging. In severe shoal areas, new channels should be created, in general, by dredging, and it may be necessary to rely on dredging for maintenance.

Diversion channels

I-43. A diversion channel may be practicable in some cases to divert upland sediment from a navigation channel or harbor. Diversion flows may be either carried around an important reach of a waterway or may be diverted completely out of the waterway by providing a separate outlet to the sea. Although diversion of river flows removes new shoal material from the waterway, certain undesirable conditions may be created. For example, current velocity may be lower as a result of the diversion, thereby reducing the sediment transporting capacity and discharge of shoal materials, originating from wave action on banks and existing areas of small depth, into the sea on ebb tide. Also, reduction of ebb discharges due to diversion of river flows may result in either a decrease or increase in the movement of material into the harbor or channel from the sea, causing corresponding changes in shoaling depending upon effect on vertical flow distribution. Obviously, any plan for diversion requires, in addition to determination of its economic feasibility, a careful analysis of its probable effects on the tidal regimen of the estuary.

Bar channels

I-44. Bar channels are located through the bar or shoal area lying off the mouth of a tidal inlet or estuary. The bar is traversed by one or more natural channels maintained by tidal flow, the largest of which is nearly always the path of the principal ebb current. Continuing encroachment of littoral sediment causes bar channels to shoal and migrate, so that natural bar channels are seldom suitable for large-ship navigation.

I-45. The planning and design of an improved bar channel should include investigation of volume, velocity, and current pattern of flows at the mouth of the estuary and over the bar; characteristics of sediment discharge; direction and location of the travel of littoral drift; direction, height, and frequency of waves which threaten the safety of vessels; history of natural bar channel locations; channel dimensions and alignment required for prospective navigation under the most severe weather conditions; and the character and use made of coastal shores on both sides of the inlet.

I-46. In selecting the alignment of a bar channel, an analysis of wave refraction in the channel region and diffraction effects of structures considered will aid in determining the most suitable location. If the best alignment for wave exposure results in increased initial dredging, the benefits must be weighed against the increased cost. The position of the bar channel at the time the improvement is being planned must not be taken as the deciding factor in channel alignment without first studying historic migratory behavior of the channel.

I-47. Since the greatest deterioriation of a bar channel is caused by moving sand, the channel should be oriented perpendicular to the largest storm waves, if possible, to lessen sand movement, to improve scouring action of tidal currents, and to provide the most favorable navigation conditions. A compromise location may be necessary if major waves approach the bar in more than one direction, or if the predominant storm direction differs from that of the largest waves.

I-48. Consideration should be given to the relative merit and economy of maintaining a bar channel by dredging as compared to use of training works before a decision is made on the most suitable plan of improvement or maintenance.

Tidal entrances

I-49. A tidal entrance is the waterway connection between the sea and an estuary, lagoon, or river entrance. There are three main types of entrances: (a) those which have rocky gorges and are not normally dependent on the tidal regime for their existence; (b) entrances at the mouths of large rivers, which are dependent on both river flow and strong tidal action; and (c) inlets, resulting from breaks in sand barriers, which are affected predominantly by wave action, littoral drift, and tidal flows.

I-50. Improvement of a tidal entrance may be desired to provide an enlarged or more stable entrance channel or to protect against wave action. Tidal hydraulic analyses are required to determine existing relations between bay and entrance characteristics, river and tidal flows, wave action, littoral drift, and river sediment. The effects associated with modification of the natural entrance by stabilization, changes in cross section and alignment, or provision of jetties must be evaluated from these analyses.

I-51. An important consideration in the evaluation is whether jetties should be provided to serve as training works for the purpose of improving the natural depth and alignment of the entrance, to regulate the currents for reasons of navigation, or to protect against wave action or shoaling by littoral drift. Jetties should not be required if the optimum relations between the desired entrance channel and the tidal regime will assure a relatively stable channel which can be maintained economically by dredging.

Artificial harbors

I-52. Artificial harbors are constructed along seacoasts in waterways which do not have natural harbors by inclosing a water area with one or more breakwaters. The principal purposes of artificial harbors are to provide refuge for vessels and small craft during storms, sheltered anchorage for the needs of recreational craft, and protected mooring for commercial craft during cargo transfer operations. The required accommodation, the adaptability of natural features, and the dictates of convenience to navigation are foremost considerations in the selection of an artificial harbor site.

I-53. The most important design problem is to locate and shape the entrance to provide the greatest protection to the harbor area against wave action and surges and the greatest safety to vessels entering during adverse weather. The opening should be located in the least exposed side of the harbor, whenever practicable. In any case, port facilities should be located in areas of minimum vertical motion, usually areas in which wave and surge amplitudes are a minimum as a result of refractive and diffractive effects, and damping. Since these effects are determined by the configuration of the reflecting boundaries, the harbor shape will have an important bearing on the damping effect on waves and surges within the basin. Thus, in a typical design study it may be necessary to consider the effects of several possible development plans, each of which implies a change in the basin's damping capacity.

Hydraulic models

I-54. Hydraulic models offer an excellent means for studying most types of tidal hydraulics problems and for evaluating the beneficial and detrimental effects of proposed physical changes. Computation methods are not available or sufficiently developed to predict the complex effects of the many interrelated factors involved in most tidal hydraulics problems. As models have displayed a satisfactory degree of similitude to their prototypes, they can be used successfully to integrate the simultaneous tidal hydraulics variables. For example, as the factors that determine the damping capacity of an artificial harbor with various proposed improvement plans are difficult to evaluate by analytical methods, model studies should be conducted for major harbors to determine the means which will minimize surge motion and to define optimum areas for specified types of operational activity.

I-55. The need for a model study should be determined from a preliminary consideration of all possible alternative solutions to determine whether the model study will be helpful in developing the likely economical plan. Comparative cost estimates may show, for example, that construction of jetties to reduce shoaling in a bar channel would greatly exceed the cost of maintaining the channel by dredging, in which case a model study would not be required.

I-56. The question of the need for a model study should not always be based on the concept that the study will result in an improvement plan which will produce major savings. An economic solution may not be found, in which case the value of the model lies in the prevention of future expenditures for ineffective plans. A model may serve to identify the basic cause of a problem which cannot be recognized otherwise, or it may indicate the most advantageous disposal area for maintenance dredging (see Chapter IX).

Prototype investigations

I-57. Prototype investigations are usually necessary for planning a waterway improvement. Analysis of existing field data and additional prototype observations, which may be required, may indicate the cause of a problem and enable determination of an economical solution without resort to model tests. If model tests are necessary, still more prototype data usually are needed for construction and verification of the model, as is discussed in Chapter IX.

I-58. In view of the hazards, time and cost involved in obtaining field measurements, particularly at the mouth of an estuary, the prototype program should be limited to those hydrographic surveys and observations of tidal heights and currents, waves, salinity, sediment, upland discharge, etc., which are essential to planning and design of the improvement. Consideration should always be given to reducing the field program by utilizing the model for obtaining data on conditions prior to improvement of the waterway. As needed measurements can be made in the model at small cost, the number of observation stations can be greatly increased to cover the problem area in more detail. Furthermore, the model observations can be made for controlled tide, waves, upland discharge, etc., which permits the collection of far more data than can be obtained in the field. Sufficient field observations are required, however, to check the model data.

I-59. Prototype observations should be taken over a sufficient period to span a representative range of conditions. Observations of tidal heights for a month may be adequate, but observations of currents, wave heights, salinity, and upland discharge may be necessary intermittently for a year or more. Consideration should be given to making additional field observations after an improvement is constructed to verify model results and to determine whether the expected benefits are being realized.

Design of Tidal Waterway Structures

I-60. Too frequently in the past, waterway structures, such as breakwaters, jetties, groins, contraction dikes, etc., have been built without benefit of adequate design, and as a result many structures have failed while others have been overbuilt to such an extent as to be wasteful of construction funds. Often an adequately designed superstructure fails because the foundation on which it stands has been inadequately protected against wave action and tidal currents.

I-61. If economically feasible, any structure exposed to wave action should be designed to withstand the effects of the highest expected storm wave. Structures located in areas in which waves may break should be designed to withstand the greater force of breaking waves instead of the essentially hydrostatic pressure of nonbreaking waves. General procedures for determining the height and direction of the design wave by use of refraction diagrams are available.

I-62. The structural design of protective works should take into account the environmental effects on construction materials. For example, if the structure will be exposed to freezing and thawing, in addition to the action of salt water, stone or timber may be preferable to concrete or steel. Availability of materials and possible construction methods should also be considered in design, as these frequently are deciding factors. Alternative types of improvement plans to reduce shoaling (e.g. contraction dikes and training walls designed as rubble mounds, walls of rock-filled steel cells, or single rows of sheet steel piling, etc.) should be investigated with a view to obtaining the most economical structure to perform the function, since construction and maintenance costs may make the difference in economic justification (i.e. favorable or unfavorable B-C ratios).

I-63. The necessity of analyzing each structure separately, considering the special conditions which may apply to each, cannot be overemphasized. As conclusions based on generalities may be fraught with chance of error, general design criteria for all conditions and locations cannot be propounded. Data must be obtained and analyzed for each individual location and problem. Knowledge of the design of similar structures elsewhere, and of the maintenance experience with them, should facilitate the analysis and aid in arriving at correct conclusions with respect to design.

Breakwaters

I-64. A breakwater is a structure employed to reflect and dissipate the force of wind-generated waves, thereby providing shelter and protection to vessels, shipping facilities, and other improvements. The two basic types of breakwaters are rubble mound and vertical wall. In addition, there are many combinations of these two types, varying as to construction material and cross section. The selection of a particular material or combination of materials and cross section are based on availability of materials, relative costs, and adaptability as pertains to conditions imposed by a given site.

I-65. A rubble mound or substantially equivalent type breakwater should be constructed when the location is in depths of water which result in attack by breaking waves, or when the necessary alignment makes an angle of less than 45 deg with the direction of the waves, as in this case the waves are not reflected but may traverse the length of the breakwater. Side slopes and stone sizes can be designed to resist the expected wave action, as the structural effects are less severe than for a vertical-wall structure. Other advantages of the rubble-mound breakwater are that it may be placed on practically any kind of foundation, and settlement of the structure results only in readjustment of component parts rather than in the incipient failure of the entire structure.

I-66. The main function of a vertical breakwater is to reflect waves, which should never be allowed to break against the structure because of the large increase in wave pressures. The angle of incidence of the waves should be less than 45 deg or they will not be reflected. Vertical breakwaters need sound foundations to ensure stability. One advantage of vertical breakwaters is that they may provide berthage for vessels.

I-67. Ideally, breakwaters should be designed to withstand the effects of the highest wave to be expected at the structure's location, but usually this will be found to be not economically feasible. General procedures for developing the height and direction of the design wave by use of refraction diagrams and the resulting wave forces on typical breakwater structures are given in Technical Report No. 4, Shore Protection Planning and Design, of the U. S. Army Engineer Coastal Engineering Research Center and the U. S. Army Corps of Engineers Engineer Manual 1110-2-2904, "Design of Breakwaters and Jetties."

Jetties

I-68. A jetty is a structure extending into a body of water, such as the oceans or main-stem waterways, from the land at the mouth of a river or entrance to a bay to aid in deepening and stabilizing the entrance channel for the benefit of navigation. The principal functions of a jetty are to direct and confine tidal flows to the entrance channel and to prevent littoral drift from shoaling the channel. Properly located jetties confine discharge areas and promote scour, resulting in greater channel depths and reduced maintenance dredging. Jetties also may protect entrance channels and inlets from storm waves and crosscurrents.

I-69. The design of jetties involves consideration of the optimum relations between number, alignment, spacing, and length to meet the needs for protection against wave action, shoaling, shore erosion, and navigation requirements. A single jetty on the updrift side of the entrance may suffice if reverses in drift direction are minor and if streamborne sediments are not important. However, where control of velocities is required to prevent shoaling by river sediments, or where littoral drift is not highly predominant in one direction, double jetties are necessary. Jetties are most often aligned parallel with the channel, as this alignment most effectively controls velocities.

I-70. Jetty spacing is governed first by navigation requirements and second by channel cross-sectional area necessary to prevent shoaling. The latter value varies with the amounts of tidal and river flows and the character of sediment involved. A "rule-of-thumb" formula for cross-sectional area between jetties, derived from naturally maintained inlets on the Pacific coast, is: cross-sectional area below mean sea level in square feet equals volume of tidal prism in acre-feet. Another formula, first proposed by M. P. O'Brien, is $A = 1000\ T^{0.85}$, where A is cross-sectional area in square feet and T is the volume of tidal prism in square mile-feet.

I-71. Jetty lengths are governed by the channel project depth and the littoral characteristics of the coastal shore. As the effective life of a jetty as a littoral barrier depends upon its impounding capacity, it is necessary to predict the rate and manner of littoral accretion in selecting the economic jetty length. The optimum jetty length also depends upon the amount of maintenance dredging which can be accomplished more economically than extending the jetty out farther into deep water.

I-72. When jetties are subject to wave action, the structural design should follow the criteria outlined for breakwaters. In addition, if tidal or littoral currents are likely to cause excessive scour along the jetty, the foundation will need to be protected to prevent undercutting and damage.

Training walls and contraction dikes

I-73. Training walls and contraction dikes are structures, which sometimes can be constructed economically, to establish or maintain interior navigation channels through shoal areas. When the natural waterway is appreciably wider and shallower than the desired navigation channel, sediment would deposit in the deeper section, as the natural forces would tend to reestablish equilibrium conditions. These contraction control works are designed to produce, or assist in establishing, self-maintaining sections of desired depth in the navigation channel. Training walls are constructed approximately parallel to the navigation channel, with the wall becoming the new shoreline, whereas contraction dikes are located nearly perpendicular to the existing shoreline.

I-74. In general, a training wall should be used in preference to contraction dikes when the affected area is small and local in character, the distance of the wall from the bank is not great, foundation conditions are favorable to wall construction, and the cost of the wall is less than an equivalent dike system. The principal consideration in designing a training wall is to ensure its stability against wave attack, foundation erosion by tidal or river currents, and any possible loading behind the wall.

I-75. Contraction dikes should be used when the reduction in waterway width is relatively large, the shoal area is quite long, and foundations are not favorable to wall construction. The dikes, which usually are constructed of stone or wood or steel piling, should be permeable in some cases to produce a milder overall effect on the natural regimen of the waterway. For example, when long dikes are required to produce an effect in the navigation channel, they may need to be permeable to prevent excessive currents and erosion around the ends of the dikes. Also, overcontraction should be avoided to minimize erosion problems.

I-76. In general, contraction dikes should extend from shore in a direction approximately perpendicular to the navigation channel to locations sufficiently near the channel to produce the desired reduction in shoaling. The spacing should be limited to the distance which results in reasonable regularity in channel alignment, depth, and width. In wide waterways, consideration should be given to the economy of constructing fewer dikes and joining their outer ends with longitudinal walls or dikes to produce the desired regularity. In an exceedingly wide section of a waterway, a series of short dikes, constructed offshore near the navigation channel, might be used to advantage to create an island which functions as the contraction works.

I-77. As reliable design criteria are not available from which the most effective and economic system of contraction dikes may be determined for characteristic waterways, for major projects model tests should be conducted to establish the best plan of contraction works.

Seawalls, revetments, groins

I-78. Seawalls, revetments, and groins are structures placed along a shoreline in a tidal waterway to protect it against erosion by wave action and tidal currents. Ordinarily, the location of these structures is established along the line landward of which further recesssion of the shore is not permitted. As these structures protect no more than the land and improvements immediately behind them, wing walls or tie-ins to adjacent land features must be provided to prevent flanking and possible progressive failure.

I-79. A seawall is constructed when there is no well-formed beach and little or no littoral drift along the shore, as along an eroding bluff, or where it is desired to maintain deep water for a quay or pier line. Although a seawall can be designed to prevent overtopping by wave action, it is not ordinarily economical to do so, and consideration should be given to lesser criteria which permit minor overtopping and damage infrequently. Construction of a seawall stops the normal process of landward erosion, but the continuing destructive forces may increase erosion depths adjacent to the wall. Thus, the wall foundation should be sufficiently deep or protected to prevent undermining.

I-80. Revetments are used to protect a receding shore due to insufficient littoral drift to maintain the shore by groins, as described subsequently herein. Both stone and concrete revetments should be considered, although when stone is readily available it usually produces the more economical revetment.

I-81. Revetments, like seawalls, need not be designed to eliminate complete overtopping as minor infrequent overtopping may not cause any serious damage. A more important design consideration is to provide sufficient depth of revetment or rock-toe protection to prevent undermining by tidal currents or currents created by the downrush of waves. In the case of concrete revetment, stepped-face seawall construction and a vertical cutoff wall provide effective means of energy dissipation and erosion protection, respectively. Stone revetment should be faced with large stone, capable of withstanding the wave forces, in the zone of wave action and smaller stone below this zone to the estimated maximum depth of erosion. The stone must be bedded on suitable layers of smaller filter stone to prevent the loss of bank or beach sand through the stone and possible revetment failure.

I-82. Groins are used to stabilize an eroding shore by compartmenting the beach and thus reducing the loss of material or by trapping littoral material. They are usually perpendicular to the shore, extending from a point landward of possible shoreline recession into water a sufficient distance to stabilize the shore. Groins may be constructed of timber, steel, stone, concrete, or combinations thereof.

I-83. In considering the use of groins for shore protection, it must be established that wave action produces a strong littoral current and a continuing supply of material from updrift of the groins. Where the supply of littoral material is small, it may be necessary to supplement the natural supply by artificially filling updrift of groins and to permit the natural supply to maintain the downdrift shore. If the supply of material is large, the amount of sand passing groins can be regulated by providing low groins which permit partial passage of material when overtopped by storm waves or by waves at high tide.

I-84. The height, length, and spacing of groins are principal design considerations. The depths in the offshore area, the extent to which it is desired to intercept the littoral stream, and the desired width of beach determine the length of groins. At the shoreward end, the top of groin level varies from normal to maximum height of wave uprush. The outer section should be approximately parallel to the slope of the foreshore which is to be maintained, and be sufficiently long to trap or hold the required amount of fill. The spacing of groins must be correlated with their length and wave direction so that when the groin area is filled to capacity the fill will extend to the base of the adjacent updrift groin.

I-85. Structural design details for typical groins are given in the U. S. Army Engineer Coastal Engineering Research Center's Technical Memorandum No. 4, Shore Protection Planning and Design.

Maintenance of Tidal Waterways

I-86. Maintenance work in tidal waterways consists of dredging channels and repairing breakwaters, jetties, and other channel and shore stabilization works. As shoaling occurs to some extent in nearly every tidal waterway, and it is most serious in those through which navigation channels have been dredged appreciably deeper than natural depths, periodic dredging is usually required to maintain project depths. Damage to tidal waterways structures may be caused by deficiencies in design, deterioration of materials, or unusual wave or current action.

I-87. If funds are not available to provide for full maintenance of all waterways, consideration must be given to the economy of limiting the work to the most urgent projects and postponing the work or partially maintaining the other waterways. In a partially maintained waterway, shipping may be possible only at reduced draft or by placing reliance on the range of tide or nonstorm periods for cargo deliveries.

Maintenance dredging

I-88. Maintenance dredging in the United States is performed generally by pipeline and hopper dredges (see Chapter VII). Pipeline dredges are used in interior channels to pump material to nearby disposal areas. The hopper dredge is used on outer bars and sometimes in interior channels when the distance to the spoil area is too long for economic operation of a pipeline dredge. The hopper dredge is also used for agitation dredging as it can operate throughout the tidal cycle by agitating on the ebb tide and resorting to conventional hopper dredge operation on the flood tide. When both the haul distance for a hopper dredge and the pipeline length are too great, it may be economical to equip the hopper dredge so that it can discharge into the bins of a rehandling facility which, in turn, pumps the material through pipelines to the spoil area. In some locations, it may prove economical to use a side-casting dredge or a bucket dredge.

I-89. The important engineering considerations pertaining to maintenance dredging concern the selection of the proper type of equipment, dredging methods, spoiling, and means of reducing the enormous volume of shoal material being removed annually. Dredging equipment and methods should be adapted to the circumstances, such as location of shoals, transport distance, availability of spoil areas, and economy of operation. There are at least three dredging practices which contribute to excessive volumes of maintenance dredging: (a) removal of low-density shoal materials, called fluff, from interior channels, (b) agitation dredging of shoal material, and (c) spoiling dredge materials in adjacent water areas, marshes, and unenclosed offshore areas along the waterway.

I-90. In some waterways, particularly along the Atlantic coast, shoal materials consist of fine clay and flocculated particles which increase in density from that of sea water to a dense mud with a thickness of 5 to 10 ft. As the upper few feet of this material is semiliquid and only slightly more viscous than sea water, it has no major adverse effect on vessels. Thus, in order to reduce the volume of maintenance dredging, current practice in some waterways is to base dredging operations on a hypothetical bottom several feet below the fluff line or to establish the bottom at a constant density level of 1100 grams per liter, leaving the lighter material in the waterway.

I-91. Agitation dredging may result in rehandling excessive volumes of shoal material, particularly when the location is so far from the mouth of the estuary that only a small percentage of agitated material reaches the sea. Instead, it settles directly back in the channel itself or in water areas adjacent to the navigation channel, and is later carried by tidal currents and wave action back into the channel, or after being carried part way to sea on the ebb tide, is returned upstream by the following flood tide with the net result that, while agitation dredging temporarily restores channel depths in localized reaches, much of the material returns to form shoals.

I-92. The point farthest upstream in a waterway from which agitated material will be carried to sea can be estimated by study of flood and ebb currents, as affected by salinity intrusions, and settling rates of the shoal material. If the time required for the material to settle to the bottom is less than the duration of ebb flow, or if ebb currents do not carry the material to sea in one tidal cycle, most of the material will settle to the bottom and/or be returned upstream by flood currents. When density currents cause a predominance of bottom flood currents, the material, as it settles, may be trapped and returned to the shoal areas. Under those rare circumstances where agitated material is transported to sea and deposited on the bars at the entrance, it is likely that some of the material returns to the waterway later.

Spoiling practices

I-93. Spoiling practices are of importance in maintaining navigation channels, as improper methods of spoiling materials from dredging operations can contribute significantly to shoaling in some waterways. Improvements in spoiling techniques usually are the least expensive of remedial maintenance measures and therefore should be the first step in any program undertaken to reduce shoaling. The basic consideration is to locate spoil areas, either outside or inside the waterway, so that dredged material does not find its way back into the navigation channel.

I-94. It has become practice to discharge pipeline spoil into adjacent marsh areas or offshore mud flats. Although marsh grasses tend to trap the coarse material, dredge flow, rainwater, wave action, and tidal currents eventually transport most of the fine material back into the navigation channel where it again forms shoals. To be effective, spoiling areas in open water or marshes must be diked to prevent runback of a large percentage of the material into the waterway, consequently requiring continuous redredging.

I-95. When the material dredged is relatively coarse sand, it may be advantageous to place pipeline spoil in shallow-water areas along the shore where current velocities and wave action are too small to transport the material. Such fills serve to contract the waterway in areas of excessive width and may be placed between spur jetties to form permanent contraction works.

I-96. In general, spoil material from hopper dredges should be dumped at sea or, if the haul distance is excessive, rehandled to shore disposal areas. The practice of placing spoil material from a hopper dredge into deep-water areas adjacent to a navigation channel should be adopted with caution. Often the material does not remain in place, as is evidenced by the fact that deep areas continue to redevelop, and it is likely that much of the spoil material is returned to the navigation channel by tidal and wave action.

Measurement of spoil material

I-97. Measurement of spoil material presents certain difficulties which make measurements of dredge quantities by some methods of doubtful accuracy. This is borne out by the indication in some waterways that the annual shoaling rate is several times the annual rate at which shoaling material can be delivered from all possible sources.

I-98. The volume of material dredged is usually deduced from before and after dredging echo-sounder surveys. It results in including, as effective maintenance, material temporarily placed in suspension and removed from shoal to other positions in the waterway. Such material may return to shoal areas so quickly as to provide very little net benefit.

I-99. Instruments are in existence that measure continuously the volume of sediment actually pumped into disposal areas, but these are not yet in general use. One method of determining dredge output is to measure the material retained in enclosed spoil areas. This requires systematic sampling and analyses of the deposited spoil, before and after surveys of the original surface receiving the fill to ascertain whether compaction has occurred, and final surveys of the fill elevation.

Maintenance of structures

I-100. Maintenance of structures should not be neglected unduly in favor of maintenance dredging. The past necessity for partial or no structural maintenance in some waterways, because of the lack of maintenance funds, is reflected in the poor condition of many breakwaters, jetties, and other protective structures. If structural maintenance is neglected, complete reconstruction may be required instead of periodic repairs of a relatively minor nature. This may be most uneconomical because of increased maintenance costs and because benefits are partially or wholly lost during the period when there is need for expensive repairs.

I-101. As damage to tidal waterway structures by wave action or tidal currents often progresses at an accelerated rate, minor repairs should be made as soon as possible to avoid serious damage. Full maintenance of interior channel regulating works is particularly desirable to assure stable navigation channels, thereby decreasing annual maintenance dredging.

Summary of Lacking Knowledge

I-102. It is evident from the tenor of discussions in this chapter that presently there is only a limited understanding of many aspects of tidal hydraulics. The engineering problems encountered in tidal water developments are complex and often baffling due to the varied interaction of many forces. The present state of knowledge does not permit the formulation of exact solutions or even mathematical expression of many factors and influences. While most tidal hydraulics subjects, such as shoaling, saltwater intrusion, etc., have been investigated in some localities, sufficient comprehensive studies have not been completed to formulate design methods and procedures for general application. Therefore, rather heavy reliance must continue to be placed on hydraulic model studies, field studies and experience, and engineering judgment in designing essential improvements.

I-103. Tidal hydraulics subjects which are deficient in knowledge and require additional investigations to reach an advanced stage of understanding are summarized as follows:

a. Tides and currents in tidal waterways:

(1) Existing theories and methods of predicting the magnitude of tides, currents, and surges in a proposed new tidal waterway, or to determine in advance the effect of major physical changes in the tidal regimen, should be more clearly defined regarding their conditions of application.

(2) The known expressions for uniform flow should be reexamined to determine the effects of density differences, varying bottom roughness, waves, etc., on their applicability to tidal waterways.

(3) More information is needed on the effects of geometry and boundary resistance on damping in cooscillating tidal systems.

b. Shoaling processes:

(1) The basic transportation laws for sediments, including muds, need to be established to predict shoaling rates and locations in proposed new channels or enlargements of existing channels.

(2) The process whereby the fine materials, composing some shoals, pass through successive stages of being in essential suspension, becoming plastic in character, and finally changing to a consolidated mass needs to be more clearly understood.

(3) Greater knowledge is necessary on the mechanics of flocculation and deposition and the effects of repetitive scour and deposition on shoaling rates.

(4) Compaction studies should systematically investigate the effects of pressure, grain-size composition, and temperature on the compaction rate of muddy deposits.

(5) More should be known of the effects of degree of mixing of salt water and fresh water on velocity distribution and turbulence and how these effects influence shoaling processes.

(6) The radioactive tracer technique should be developed further and used more widely to identify the source of materials comprising shoals and to establish the relative importance of various shoal materials and sources.

(7) Improved knowledge of the shoaling process and transportation of estuarine sediments should be examined with a view to more efficient use of dredging equipment.

c. Saltwater intrusion:

(1) There is need for more complete knowledge of the characteristics of the saltwater wedge, the flow conditions around it during its advance and retreat, and its effect on newly deposited sediment and that still in suspension.

(2) An appraisal is needed of the significance of physical features and hydraulic regimen of estuaries on the extent of salinity intrusion.

(3) The effect of predominance of upstream flow due to saline intrusions on the shoaling process must be more clearly evaluated.

(4) It should be determined whether shoaling occurs in the region where sediment first encounters the saline water (effect of salt water on flocculation phenomena) regardless of hydraulic characteristics which otherwise might keep the sediment moving seaward.

d. Upland discharge:

(1) Criteria are needed which will permit determination of the freshwater discharge into an estuary necessary to hold saltwater intrusion to a given desired location.

(2) More knowledge is needed on the effect of artificial changes in upland discharge on tidal hydraulic and shoaling characteristics of estuaries.

e. Navigation channels and structures:

(1) Improved criteria are needed to indicate the effects of any plan of improvement on salinity, shoaling, tidal circulation, and pollution in an estuary.

(2) Criteria concerning relatively stable cross sections of channels subject to scour and deposit must be determined to predict long-range adjustments which follow artificial interference with natural channels.

(3) Practical methods of modifying waterways should be investigated to determine whether shoaling can be made to occur in the vicinity of disposal areas of large potential capacity.

(4) Although general criteria are available, increased tidal hydraulics knowledge should permit the development of improved criteria for the planning and design of channels and structures, resulting in greater hydraulic efficiency and reduced maintenance costs.

f. Tidal entrances:

(1) Improved criteria are needed to establish relations between tidal prism, cross-sectional area, flow velocities, wave action, sedimentation characteristics, bar location, and geometry of tidal entrances to assure relatively stable conditions.

(2) More information is needed on the effects of adjacent shores and littoral drift on stability and shoaling of entrances.

g. Navigation requirements:

(1) Determination should be made of the effects of soft bottoms containing fluff material on vessel squat, maneuverability, and machinery.

(2) Bases are needed for establishing economical navigation depths through fluff material.

(3) Improved criteria are needed for establishing economical depths, widths, and turn characteristics of navigation channels.

h. Maintenance dredging:

(1) Basic data should be obtained and analyzed to determine the efficacy of agitation dredging operations under various tidal hydraulic conditions. This should include the settling rates of agitated shoal material.

(2) The use of flocculents should be explored for controlling deposition and consolidating fluff for the purpose of increasing the effectiveness of dredging.

(3) All other maintenance dredging practices should be constantly reviewed to ensure the permanent removal, consistent with economy of operations, of the maximum volume of sediment from waterways.

i. Prototype investigations:

(1) More field measurements and analyses of data obtained therefrom are needed to verify existing theories and model tests and for the purpose of developing new or improved criteria.

(2) More determinations of the undisturbed dry density of shoals need to be made.

(3) More accurate means of measuring the inflow of sediment to an estuary should be developed.

j. Instrumentation:

(1) More research is required for the development of new, or the improvement of existing, methods of sampling muddy sediment deposits, transportation rates, and concentrations.

(2) There is a need for the development of instruments capable of recording turbidity of flow, in-place density of shoal material, and amount of solids being pumped through dredge pipelines.

CHAPTER II
COMPUTATION OF TIDES AND CURRENTS

by

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II-1. It is not necessary to discuss herein the basic theories of the tidal phenomena experienced in the oceans of the world, for they are environmental factors that cannot be altered. However, the characteristics of the ocean tides are impressed on the undulations that they generate in tidal waterways, and it is therefore necessary to examine briefly the characteristics of the tides of the oceans.

II-2. The tide is not a constant phenomenon throughout the oceans; instead, it varies greatly both as to amplitude and as to its other important characteristics. Disregarding the disturbing effects of meteorological forces, the mean range of tide along the shorelines of the open ocean is as little as 0.5 ft and as much as 30 ft, depending on location. There are three distinct types of tide: the diurnal, the semidiurnal, and the mixed tide. The diurnal tide has but one high and one low tide per lunar day; the semidiurnal tide has two nearly equal high tides and two nearly equal low tides per lunar day; and the mixed tide has two markedly unequal high waters and two markedly unequal low waters per lunar day.

II-3. These tides generate an undulation in tidal waterways, and once started, it propagates upstream to some point where further progress is terminated by a barrier, or to where the accumulating attrition causes the undulation to disappear. If the length of the estuary exceeds the length of the tide wave, as in the case of the Amazon River in South America, the system may contain two or more tides at the same time. Thus, the tide may be rising or falling in two or more reaches of the estuary at the same time. In some situations, the geometry of the waterway causes a stationary wave, but these cases are not so frequently encountered as the so-called progressive waves. Progressive waves travel upstream with a celerity,[1*] under frictionless conditions, of:

$$c = \sqrt{g\,(d + H)} \qquad \text{(II-1)}$$

where

c = celerity in feet per second

g = acceleration due to gravity in feet per second per second

* Raised numerals refer to similarly numbered items in Literature Cited at the end of this chapter.

d = depth in feet at mean low water

H = stage of tide above mean low water

II-4. During the propagation of a progressive wave, the high-water portions travel faster than the low-water portions of the wave, and this helps to distort the shape of the wave. As the wave progresses up the estuary, the duration of the rise decreases and the rate of rise increases; conversely, the duration of fall increases and the rate of fall decreases. The shape of a curve representing tidal heights plotted against time shifts from that approximating a sine or cosine curve to that exhibiting a quick rise of relatively short duration followed by a slow fall of relatively long duration. These shapes are illustrated in fig. II-1 which also shows the times of low and high water as observed and as computed by the use of equation II-1. In connection with the latter, it will be observed that the computed times are earlier than the observed, indicating that the speeds computed are greater than those actually observed in the estuary. The differences are due in part to the obvious fact that the propagation up the estuary is not frictionless, and also to modifications of the basic wave caused by reflections.

II-5. Under certain circumstances, specifically where the depths are relatively much greater at the higher stages of tide than at the lower stages, the celerity of the higher portions of the wave may greatly exceed that of the lower portions. For example, the celerity of the trough portion of a tide wave having a range of 20 ft in traversing a waterway having a mean depth of 10 ft at low tide would be:

$$c = \sqrt{32.2\ (10 + 0)} = 18 \text{ fps}$$

The celerity of the crest of the same wave would be:

$$c = \sqrt{32.2\ (10 + 20)} = 31 \text{ fps}$$

The higher portion of the tide wave overtakes the lower and a traveling jump[2] occurs. This phenomenon is called a tidal bore, and is known[3] to exist in such waterways as the Hooghly (India), the Amazon (Brazil), Turnagain Arm at the head of Cook Inlet (Alaska), and in the Bay of Fundy's shallow estuaries (Canada), to name a few. Bores have a very steep front, rising 6 ft or more virtually instantly, followed by a flatter but still rapid rise. The bore at any given locality is quite a variable phenomenon; during a period when the neap tidal ranges are occurring, the bore may not appear at all, while during the spring tides it attains its greatest heights and severity. The speed of travel may be as much as 15 mph. The larger and swifter bores seriously menace navigation.

II-6. The derived tide in a waterway is greatly affected by resistances, the

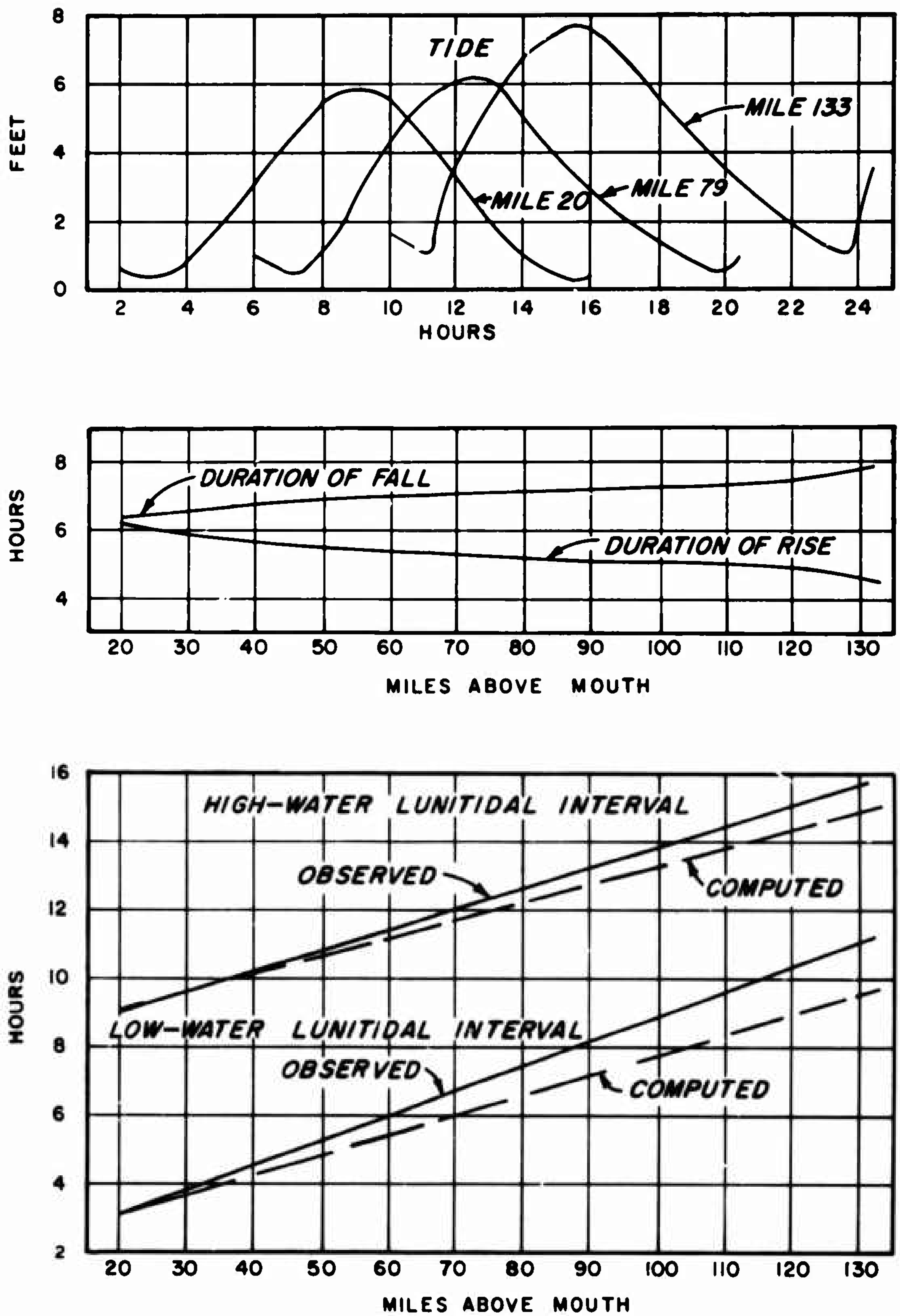


Fig. II-1. Tidal rise and fall data on the Delaware estuary geometry of the waterway, local winds, and discharges of fresh water from the upland. The tide range may increase, decrease, or remain the same as the tide ascends the waterway. The shape of the wave may become much distorted from the sinusoidal shape that it may have had at the mouth. This is due to: (a) the retardation of the rate of travel (as already discussed), (b) reflections, or the so-called shallow-water components of the observed tide, (c) upland discharges of fresh water, (d) winds. Changes in either the range or the shape (or both) of the tide have important consequences from the viewpoint of navigation.

II-7. As stated above, the tidal range may increase, decrease, or remain essentially constant as the wave ascends the estuary. Fig. II-2 shows that the range of tide in the Delaware estuary alternately increases, decreases, and increases while the cross-sectional area decreases in the so-called classical funnel-shaped manner. The Hooghly River tidal range decreases rapidly as the tide proceeds upstream, although the cross-sectional area decreases as in the Delaware. Fig. II-3 shows the tidal range in the Hudson River decreasing by about 40 percent in the first half of its length, then increasing to a value greater than that at the mouth while the cross-sectional area fluctuates in the downstream half and decreases fairly uniformly in the upstream half. This figure also shows that the range of tide in the Vicksburg salinity flume[4] increased considerably as it propagated upstream, although the cross-sectional area is constant.

II-8. Clearly, cross-sectional area is not the sole factor producing modifications of the range of tide as the wave propagates upstream. In fact, the range is decreased by friction, either increased or decreased by reflections and the interference of harmonics known as overtides, and increased by the convergency of the cross-sectional area in the upstream direction. In most cases, all of these factors may be operating concurrently to modify the tide wave as it travels upstream. Returning to fig. II-2, the decrease of tidal range in the Delaware between miles 40 and 60 is not due to cross-sectional area, width, or mean depth deficiencies or excesses; the first two of these factors decrease fairly uniformly upstream in this reach of the estuary, while the depth is fairly constant. However, there are two large islands in the reach, and it is probable that they cause reflections that modify the primary wave. Upstream of mile 60, the range of tide increases fairly uniformly while the cross-sectional area decreases equally uniformly. Evidently the convergency proceeds at a rate that more than counterbalances the effect of friction. However, the tidal range at the head of tide in the Delaware has not always been greater than the range of tide at the mouth. In its unimproved state, depths were much less than those now in existence as a result of dredging and rock removal, and the friction was evidently greater. Thus, although the cross-sectional areas converged rapidly toward the head of tide

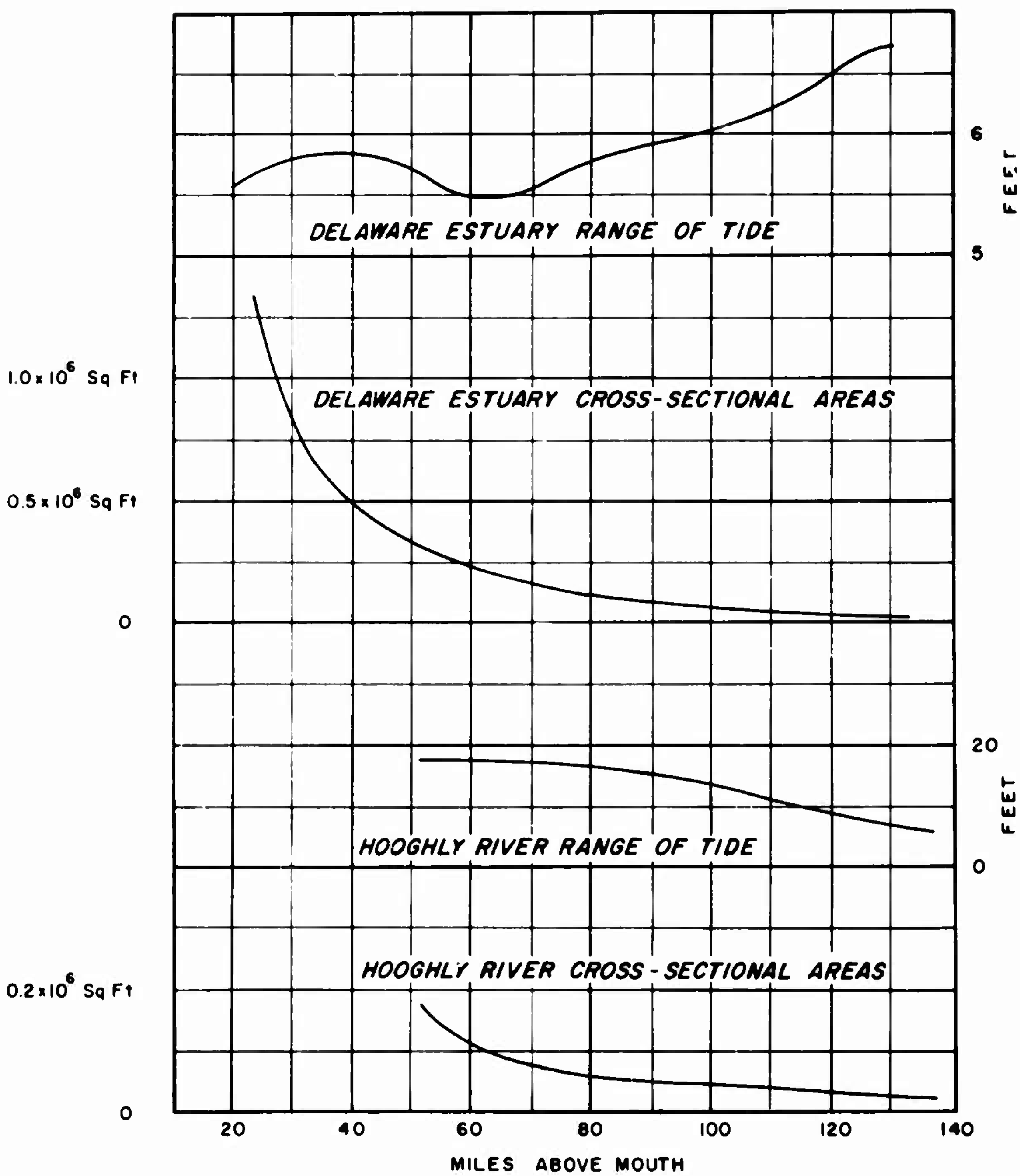
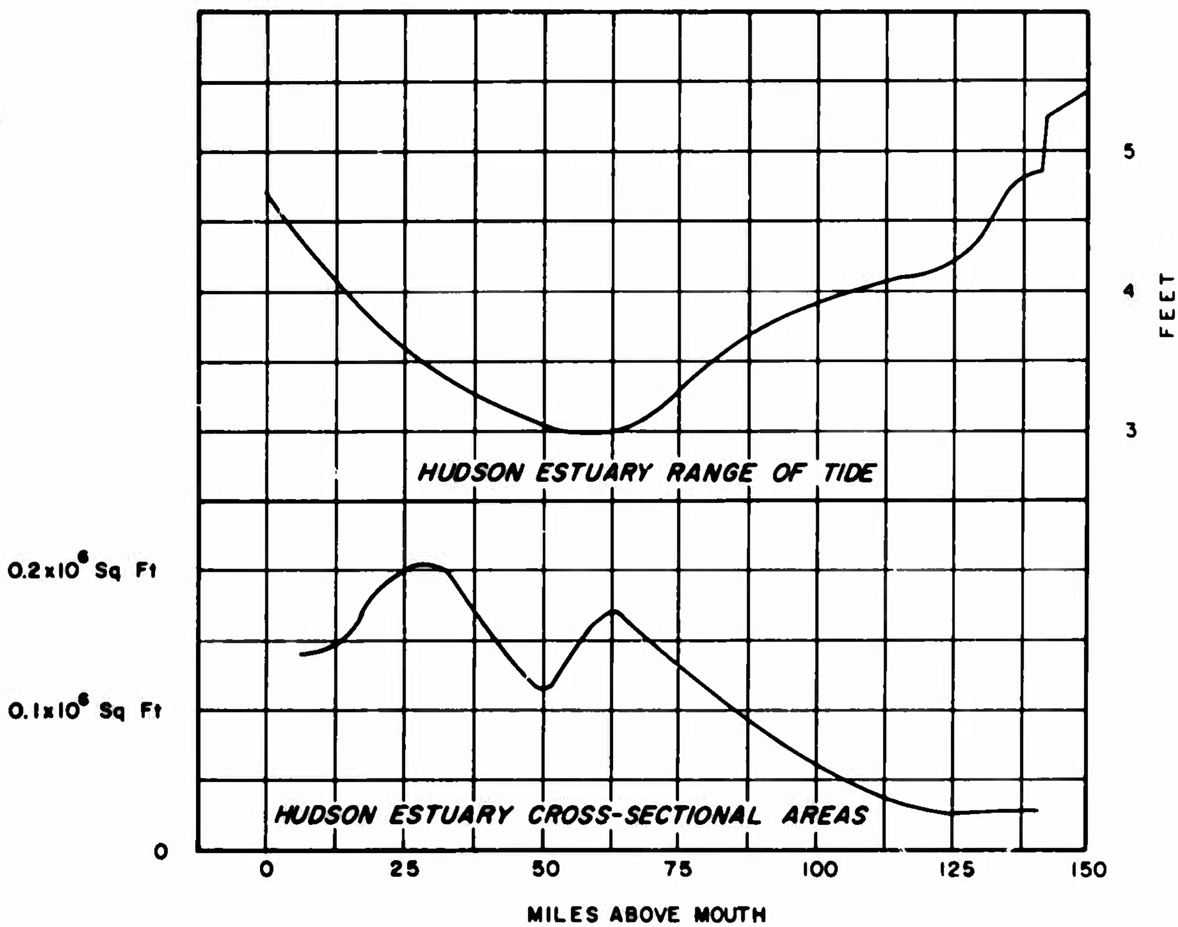


Fig. II-2. Comparison of tides and cross-sectional areas, Delaware estuary and Hooghly River



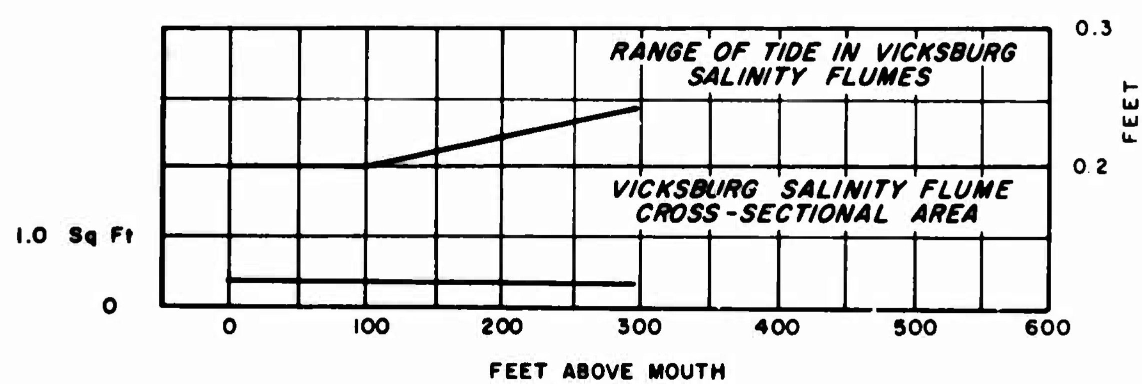


Fig. II-3. Comparison of tides and cross-sectional areas, Hudson estuary and Vicksburg salinity flume before the river above mile 100 was dredged, the range of tide decreased because the frictional losses were not counterbalanced by the convergency.

II-9. In the case of the Hooghly, the river contains ten shallow bars, or crossings, and it curves frequently. At low tide especially, when depths are much less than at high tide because of the large variation of water surface elevation between high tide and low tide, frictional resistances are large and they reduce the amplitude of the tide wave as it ascends the estuary much more rapidly than it is increased due to the convergency of the cross-sectional area. It is also to be noted that the rate of convergency is about half that of the Delaware. With respect to the data shown in fig. II-3, it is likely that the considerable fluctuations of cross-sectional area in the Hudson between the mouth and mile 65 cause excessive losses of energy, and that they generate reflections that modify the primary wave. Upstream of mile 65, the range increases at a fairly uniform rate and the cross-sectional areas decrease uniformly. It is apparent that the rate of convergency in this reach of the river more than offsets the frictional losses. In the case of the Vicksburg salinity flume, it is somewhat remarkable that the range increases considerably in the face of a constant cross-sectional area. It can be shown that there are reflections in this flume that combine with the primary wave and produce the larger resultant wave that has been observed.

II-10. In the case of the Vicksburg flume, theory and computational procedures that will be discussed in detail later show that the primary tide that was generated at the "ocean" end of the flume was in fact somewhat greater in amplitude than that which was observed. The observed tide was the resultant of the primary tide that was generated and a reflection that moved back through the flume and combined with the primary wave. The primary wave, as generated in the "ocean," propagated up the flume, losing amplitude as it progressed, until it reached the end of the flume; it was then reflected and traveled towards the ocean, also suffering loss of amplitude as it progressed. Fig. II-4 shows the rate of loss of amplitude of the primary and reflected waves as they move respectively towards the head of the flume and the "ocean"; the figure also shows the times of high tides of the primary and reflected waves as they occur at several points in the flume. These graphs show that, although the amplitude of the primary and reflected waves at station 200, for example, are respectively 0.078 and 0.049 ft, they do not combine to produce a resultant wave of 0.127 ft because they are out of phase; the high tide of the primary wave occurs at 60 sec, while the high tide of the reflected wave occurs at about 140 sec of the 600-sec cycle. Also shown is the manner in which the two tides combine to produce the resultant tide with an amplitude of 0.11 ft. This causes a range of 0.22 ft, which is exactly that observed in the flume. The times of high and low tides are at 90 and 390 sec, as

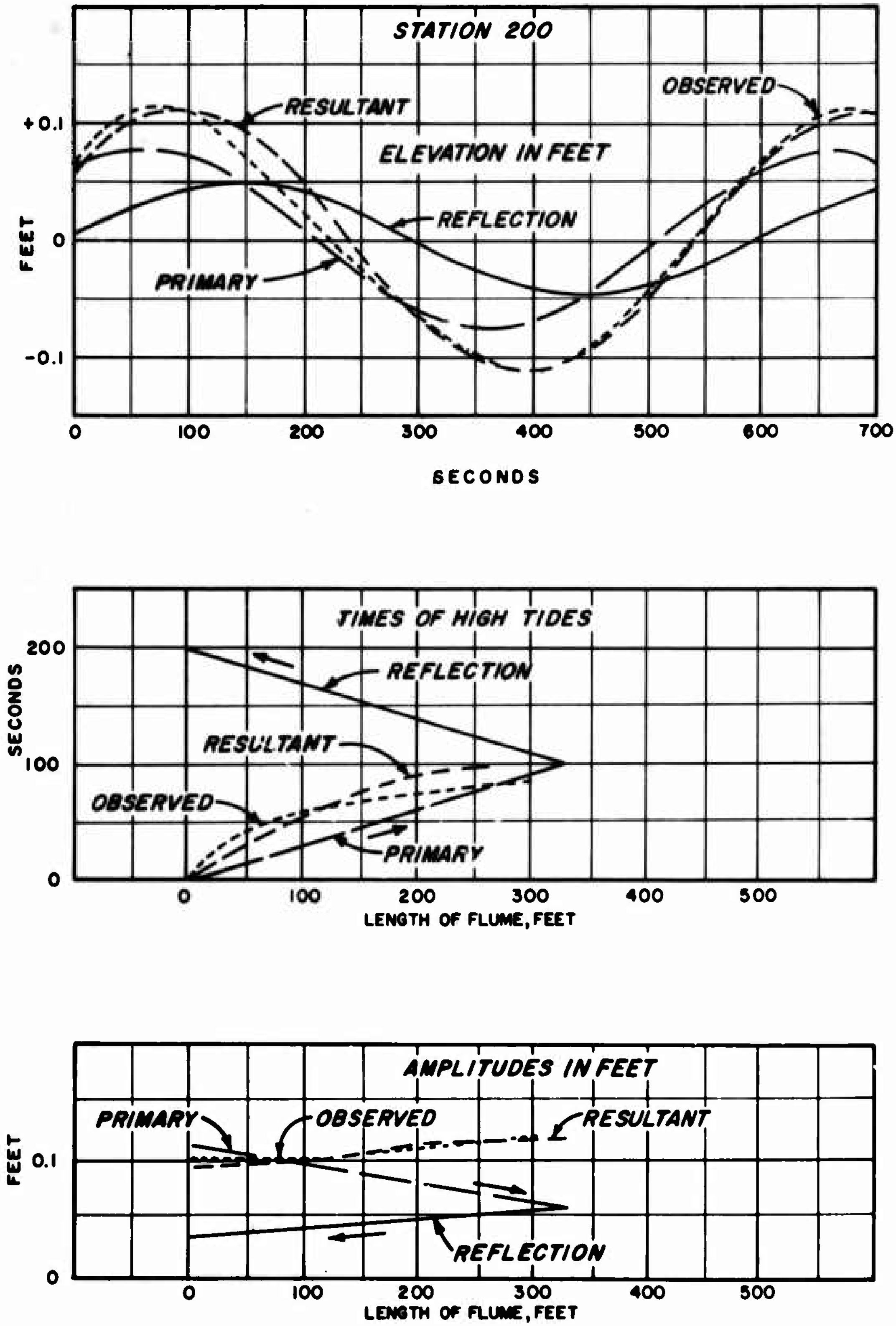


Fig. II-4. Vicksburg salinity flume tides compared with observations of 72 and 400 sec. The observed data are plotted to facilitate comparisons with the computed values.

II-11. It is seen that the tide in the flume depends upon the primary tide at the "ocean" end, the friction that causes losses of amplitude of the undulations, the depths of water which govern the celerity with which the undulations propagate, and the length of the flume. If the primary wave at the entrance had a larger amplitude than that which has been under discussion herein, its amplitude throughout the length of the flume would have been greater. Accordingly, the reflection that travels in the opposite direction would have had greater amplitudes, and it follows that the resultant would have been greater. Similarly, lower frictional losses would have caused the primary wave to lose less amplitude as it traversed the flume, and the reflection and the resultant waves would have been greater. Conversely, a smaller primary wave at the entrance or a higher frictional loss rate would have caused lower primary, reflected, and resultant waves all along the flume. If the depth of water had been greater or shallower, the undulations would have been higher or lower than in the case mentioned previously, and the phasing of the primary and reflection waves would have been different. Turning to the upper set of curves in fig. II-4, it is seen that the high water of the resultant undulation would have been 0.125 ft if the reflection had arrived 100 sec earlier relative to the time of the primary wave, and 0.076 ft if the reflection had reached station 200 the same amount later. Such changes in the phasing of the primary and reflected waves could occur as a result of combinations of changes of length and changes of depth.

II-12. The theory applied in computing the results described generally above is based on the fact that the tide at the entrance produces a forced oscillation within the flume having a period that coincides with that of the generating tide. This condition is known as cooscillation. If it is assumed that a plot of heights of the generating tide against their times of occurrence is a cosine curve (and this assumption is satisfactory for open ocean tides that have not been disturbed by winds and other nonastronomic forces), its equation is of the form:

$$\eta = A \cos \frac{2\pi t}{T} \qquad \text{(II-2)}$$

where

η = the elevation of the free surface at time t

A = the amplitude of the tide (which is one-half the range)

t = the time in seconds

T = the tidal period in seconds

At any given time t, the undulation within the flume (ignoring friction) has the form:

$$\eta = A \cos 2\pi \left(\frac{t}{T} - \frac{x}{L}\right) \qquad \text{(II-3)}$$

where

x = distance along the flume with x = 0 at the "ocean" entrance

L = the wave length

This equation more conventionally is written:

$$\eta = A \cos (\sigma t - \kappa x) \qquad \text{(II-3.1)}$$

which may be obtained from equation II-3 by replacing $\frac{2\pi}{T}$ with σ and $\frac{2\pi}{L}$ with κ. These terms are designated "tidal frequency" and "wave number," respectively, by Ippen and Harleman.[7] As this progressive wave travels up the flume, it suffers attrition due to friction. The amount of energy dissipated is proportional to the total energy of the wave, and consequently the friction leads to a logarithmic decrease of amplitude in this constant-depth flume. The amplitude of the wave at any given distance x within the flume (x = 0 at the "ocean" entrance) is then (Fjeldstad,[5] 1929):

$$\eta = A e^{-\mu x} \cos (\sigma t - \kappa x) \qquad \text{(II-4)}$$

where

μ = a coefficient of damping, or damping modulus

The end of the flume is a fixed barrier, and the wave is reflected back upon itself[6] without change in elevations and depressions. This reflected wave combines with the primary wave at the barrier and produces a resultant wave with an amplitude exactly twice that of the primary at that point. This is shown in the bottom set of curves in fig. II-4. The reflected wave travels in the minus x direction but its amplitude is computed by the use of equation II-4, as in the case of the primary wave, except that the distance x is measured from the fixed barrier.

II-13. It is of course apparent that the salinity flume is an unusually simple case; the resolution of the observed tide into its components was accomplished easily by taking advantage of the fact that the heights of the observed tide at the head of the flume are exactly twice those of the primary undulation. Knowing the amplitude of the primary wave at the head of the flume permits computation of the heights of the primary wave at any location at any time t by employing equation II-4 with the amplitude A of the primary wave at the head of the flume and the distances x as negative. The heights of the reflection are similarly computed with the distances x as positive. The primary and reflection wave heights are added algebraically to obtain the resultant. The values of the damping coefficient and the wave number $2\pi/L$ are obtained experimentally.[7]

II-14. A closed-end tidal channel of uniform depth and width throughout its course is rarely encountered. Most natural tidal waterways have erratic variations of depth and width, and their sections are frequently cut up by islands and shoals. Furthermore, they usually receive freshwater discharges, and these have their effects on the tidal establishments. Tidal canals usually do not terminate at a barrier, but instead connect two tidal bodies, or perhaps extend from one body having a tide to a tideless basin. Thus, most natural tidal waterways have tides that are the resultants of a primary tide and a number of reflections caused by abrupt changes in width and depth. However, there may be cases where there are no appreciable reflections when the changes in width and depth are gradual instead of abrupt,[6] and where the waterway is of such length that the primary wave gradually vanishes before reaching a barrier that would cause a reflection. Most tidal canals have a tide that is the resultant of two independent tidal waves propagating in opposite directions.

II-15. Harleman and Ippen[8] succeeded in applying equation II-4 to the tides of the Bay of Fundy in connection with considerations as to the possible effects of a proposed tidal power project at Passamaquoddy. This bay does not have uniform depth and width, but its sections apparently vary gradually from mouth to head of tide, and therefore the only reflection of consequence is that from the head of tide. Accordingly, the authors of the referenced study proceeded to evaluate the damping proportionality factors by plotting observed values of tidal amplitudes against the time angle σt_H (where t_H is time of local high tide at various locations). Having these values, and assuming that there were no significant additional reflections, it was possible to produce an entirely satisfactory analysis of the observed tides. However, it is to be noted that it was necessary to have tidal observations before suitable values of μ and κ could be obtained.

II-16. Tidal current velocities for a uniform channel can be computed by employing an equation developed in accordance with assumptions used in the derivation of equation II-4. This may be conveniently accomplished by combining the continuity equation with equation II-4, and similarly with the expression for the reflected wave, and thereby produce the following, according to Harleman and Ippen.[8]

$$u = \frac{A\sigma}{h} \frac{1}{\sqrt{\mu^2 + \kappa^2}} \left[e^{-\mu x} \cos(\sigma t - \kappa x + \sigma) - e^{\mu x} \cos(\sigma t + \kappa x + \alpha) \right] \qquad \text{(II-5)}$$

where

u = the resultant current velocity due to the primary and the reflected wave

h = the depth at mean sea level

$\alpha = \tan^{-1}\left(\frac{\mu}{\kappa}\right)$

II-17. Procedures more generally applicable than those discussed in the preceding paragraphs have been developed as follows. In most open channel flows, one makes the assumptions that slopes are small, all significant flow velocities are horizontal (vertical velocity components negligible), that the flows occur in the longitudinal direction along the course of the channel, and that flow from the sides of the waterway towards the midsection, or vice versa, are negligible. The differential equations of the flow in any such channel are those of any non-steady flow in an open channel. They are the equations of continuity and of momentum. The continuity equation merely says that, for an incompressible fluid in motion in a channel, the quantity that enters a given section in a given time plus or minus the quantity stored or evacuated from that section in the same time must equal the quantity that leaves the section. In its simplest form[2] the continuity equation is:

$$\frac{\partial Q}{\partial x} + b\frac{\partial H}{\partial t} = 0 \qquad \text{(II-6)}$$

where

Q = discharge

x = distance along the waterway

b = width of section

H = elevation of water level above datum

t = time

The momentum equation is usually expressed as follows:

$$\frac{\partial H}{\partial x} + \frac{v}{g}\frac{\partial v}{\partial x} + \frac{1}{g}\frac{\partial v}{\partial t} + \frac{v^2}{C^2 R^2} = 0 \qquad \text{(II-7)}$$

where

v = velocity

C = Chezy coefficient

R = hydraulic radius

II-18. According to Dronkers,[9,12] there are several methods for solving these equations. These include the harmonic methods, direct methods, and the so-called method of characteristics. The harmonic methods involve resolution of the composite tidal motion into its components by Fourier analyses. These components can then be treated separately. The direct methods include the finite difference methods, the power series method, and the iterative method. In essence, these methods subject equations II-6 and II-7 to some process of numerical integration. The method of characteristics involves analyses of the propagation of the tidal waves on the basis of the characteristic elements of the differential equations. In addition to these, the method developed by Pillsbury[3] apparently has merit, although it is perhaps not as satisfactory from the viewpoint of a mathematician.

II-19. In applying any of the methods, it is necessary to "schematize" the waterway,[2] as most tidal bodies are irregular. Every irregularity, such as shoals, piers, bridges, islands, etc., has its effect on the local flow. The first schematization divides the channel into two portions: that which conveys as well as stores, and that which stores but does not convey. A section of the waterway where there is a pronounced cove would have a channel that conveys and stores, and the cove, which stores but does not convey. Next, it is necessary to divide the waterway into sections of such length that the widths and depths within each section are reasonably constant. This does not usually require a very large number of sections; it is necessary, however, that the length of each section be small compared with the wave length of the tide. According to Dronkers-Schonfeld, practice in the Netherlands results in sections of 5 to 10 kilometers in length.

II-20. The next problem in applying the calculation methods is the proper method to linearize the frictional resistance. The quadratic nature of this term in equation II-7 had been one of the main difficulties in making the computations. Procedures conceived by Levy (1898) and Parsons (1918) and further developed by Lorentz (1926) and Mazure (1937) resulted in the following expression:

$$r = w \frac{8}{3\pi} |Q| = 0.85\ w\ |Q| \qquad \text{(II-9)}$$

where

r = a linear resistance factor

$w = \frac{1}{K^2}$ where K is the Bahkmeteff conveyance factor

Pillsbury notes the quadratic character of the Chezy "C" as determined by the Kutter formula, in which it is premised that the frictional resistance depends on the slope of the water surface and thereby varies as the square of the current velocity. To linearize the computations, he suggests that the Chezy "C" be computed by the Manning formula:

$$C = \frac{1.486}{n} r^{1/6}$$

Ippen[10] accomplishes the same purpose by converting the Chezy "C" into the expression:

$$C = \sqrt{\frac{8g}{f}}$$

where

f = Darcy resistance coefficient

II-21. Selection of the proper values for the Chezy "C," the Manning "n," or the Darcy "f" is somewhat more difficult for tidal waterways than for upland nontidal streams. Observations of the accuracy desirable to permit computation of the proper value are much more involved, but nevertheless some observational data must be obtained in order to avoid serious error.[10]

II-22. In the Netherlands,[2] careful schematization permits them to keep "C" from section to section relatively constant. Although by theory "C" should be varied as a function of time as well as by section, they do not make this refinement, although considerable error in predictions at times near slack water thereby occurs. They point out, however, that such errors are of little practical importance and hence relatively great errors in "C" are permissible then. It is stated that good results are obtained with a value of C = 50 in the metric system (90 in U.S. measures).

II-23. A number of applications of the various computational procedures have been made. A few of these are presented below.

Model Sea Level Panama Canal

	Midpoint Tides and Currents			
	Observed	Lamoen*	Einstein**	Pillsbury†
Height of high tide	+4.9 ft	+5.2 ft	+6.4 ft	+4.9 ft
Time of high tide	0.0 hr	0.0 hr	0.0 hr	0.0 hr
Height of low tide	-4.5 ft	-5.5 ft	-6.2 ft	-4.2 ft
Time of low tide	6.0 hr	5.5 hr	6.0 hr	6.0 hr
Max velocity to Atlantic	4.1 knots	3.6 knots	4.2 knots	3.8 knots
Time of max velocity	1.0 hr	1.3 hr	1.4 hr	1.0 hr
Max velocity to Pacific	3.8 knots	3.7 knots	4.2 knots	3.5 knots
Time of max velocity	7.5 hr	7.0 hr	7.3 hr	7.0 hr
Time of slack after flow towards Atlantic	4.4 hr	4.7 hr	4.3 hr	4.5 hr
Time of slack after flow towards Pacific	10.4 hr	12.0 hr	10.3 hr	8.5 hr

* Lamoen[11] computed his values by a technique developed by Holsters, which may be considered as a modified application of the principle of finite difference integration.

** Einstein's computations for the Committee on Tidal Hydraulics are based on his so-called linear theory.

† Pillsbury's values are obtained by one of the direct methods.

Dronkers[9] computed the tides and currents of Tampa Bay as an example of the application of the harmonic method. The results are summarized below.

Tides

	Time of Propagation, min. from St. Petersburg, 3-4 September 1955		
	Egmont Key	Tampa	Safety Harbor
Computed	-150	+25	+100
Observed	-145	+15	+105

	Amplitudes, ft		
	Egmont Key	Tampa	Safety Harbor
Computed	1.25	1.00	1.05
Observed	0.95	1.20	1.25

Currents

	Egmont Key	Tampa	Safety Harbor
Ebb velocity, fps			
Computed	2.60	0.95	1.85
Observed	2.5	0.8	1.8
Flood velocity, fps			
Computed	1.70	0.60	1.00
Observed	1.6	0.7	1.3

Mention has already been made of the application of equation II-4 to the Bay of Fundy by Harleman and Ippen.[8] In this work, calculations were made of the tide at Eastport for comparison with an observation and the results are coplotted with the observed graph of height versus time; the computed points are so close to the observed tide as to be virtually undistinguishable. Current velocities were computed at three locations where observations were available. These are compared below:

Location	Flood Current, knots Observed	Flood Current, knots Computed	Ebb Current, knots Observed	Ebb Current, knots Computed
Eastport	1.5	2.8	1.5	2.8
Cutler-Grand Passage				
Location a	3.3	3.1	3.3	3.1
Location b	2.4		2.4	

II-24. These results show that satisfactory methods for computing the tides and currents exist, although the above comparisons for Eastport are not impressive. It appears that all of them are capable of producing reasonably accurate results with some difficulty, but the effort is not prohibitive after the procedures are understood. At present, only the Pillsbury technique has been reduced to language that the average engineer can comprehend; all of the other methods are described in mathematical language that is difficult for one not accustomed to handling differential equations. There can be no question that these methods should be brought into the realm of practicing engineers, and that a suitable presentation as to the particular applications best suited to each method is desirable.

II-25. Reference 2 concludes with a comparative appreciation of the computation methods available. In brief, the authors state that the harmonic methods are particularly well suited for studies of periodic tides, but not for nonperiodic motions, such as storm tides. The linearization of the resistance can be improved by successive approximations, but an experienced computer can often make a fair estimate at once. The harmonic methods are very appropriate for reconnaissance investigations. The so-called direct methods require the simultaneous solution of a number of nonlinear equations; this is not especially difficult for an uncomplicated waterway, but when the system requires many sections in its schematization, trial and error procedures are necessary and they may become prohibitively difficult. The characteristics methods require much more labor to apply than the harmonic methods and the direct methods for relatively simple waterways. One of the greatest virtues of the characteristics methods is its straightforwardness for studies of very complicated systems. The characteristics methods are especially well suited for studies of waves of finite extent produced by lock operations, dam failures, etc. The authors comment on the use of electronic computers for making tidal computations. They observe that the efficiency of these machines depends on the programming of the computations in such manner that they work uninterruptedly for a considerable period of time. Tidal computations do not adapt themselves very well to this kind of procedure. Further, it has been observed that a considerable part of the time required for the investigation is taken up by the work of finding the best schematization of the waterway, and this cannot be accomplished by a computer. When the requisite experience is available for performing this schematization study, the same team of experienced people can quickly accomplish the actual computations.

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CHAPTER III
SEDIMENTATION IN TIDAL WATERWAYS

by

C. F. Wicker and R. O. Eaton

Introduction

III-1. The sedimentation that is taking place in tidal waterways under existing geological, meteorological, and oceanographic conditions not only causes navigation channels therein to shoal, but produces progressive deterioration of the waterway itself. Wide bays become marshy areas threaded by a maze of interconnecting shallow passages. Estuaries gradually fill outward from the banks and ultimately become expanses of marshes traversed by a meandering stream carrying the upland discharge to sea. These results do not necessarily require a geologic age for their accomplishment. It is a matter of record that numerous small tidal waterways that were of great value to the colonists of what is now the eastern seaboard of the United States are now mud flats. According to the Soil Conservation Service, due to overgrazing and cultivation of land in the contributing watershed, the present rate of deterioration of our bays and estuaries exceeds that of the early days of our history.

III-2. Although this long-term deterioration of tidal bays and estuaries may represent an ultimately greater economic loss than that represented by the present and prospective average annual cost of maintaining navigation channels in these bodies, the latter constitutes the immediate problem and is the subject discussed herein. Fortunately, channel maintenance if properly planned is compatible with preservation of the waterway itself.

Sources of Sediments

III-3. The principal sources of sediments that shoal navigation channels are:

a. The land areas drained into the tidal body.

b. The bed of the tidal waterway itself.

c. The littoral drift in motion along ocean beaches adjacent to the mouth of the tidal waterway.

d. Sanitary and industrial wastes.

e. Marine life in the tidal waterways.

f. Windblown material.

g. Improperly deposited spoil from channel dredging operations.

III-4. All rivers and streams transport detritus at rates dependent on many factors. Nature has been working hard to level the land masses and her principal tool is water. Her success in the distant past is measurable by the thick strata of such sedimentary rocks as shale, sandstone, and the conglomerates. Evidence of the effectiveness of her work in the recent past and at present is found in the masses of unconsolidated deposits of silt, sand, and gravel. In the opinion of many observers, man has increased the burden of solids carried by the rivers to the tidal waterways by forest clearing, improper agricultural practices, overgrazing, and discharges of waste into the rivers.

III-5. The rate of supply of upland material depends upon many factors: the type of soil, slope steepness, ground cover, freezing and thawing, distribution and amount of precipitation, the hydraulic characteristics of the stream or river, and the activities of man within the watershed, including pollutant introduction. Obviously, a watershed having a light soil will erode more rapidly during heavy rainfall than one with a heavy soil or with considerable rock outcroppings or cover of shingle and boulders. Steep slopes cause more rapid flow of water and consequently experience greater erosive power. Ground covers, including the grasses, bind the soil together with their roots and tend to retard the rate of flow across the surface. Freezing causes expansion and cracking, and after thawing the soil is in a condition much more susceptible to erosion. Intense rainfalls cause much more rapid flows across the surface than gentle precipitation. Even the actual impact of large raindrops causes soil erosion. Accordingly, erosion will be greater in a locality where frequent intense rainfall is experienced than in one having an equitable distribution of precipitation throughout the year. Streams with steep slopes flow more swiftly than those with slight gradients and are more competent to carry sediments. Solid pollutants introduced by man include sewage, industrial wastes, and mine debris. In some cases, the total load dumped into the stream by man may exceed the quantity reaching it as a result of natural processes.

III-6. Another source of the sediments that form shoals in navigation channels in tidal bodies is the bed of the waterway itself. In a great many cases, the width of the established navigation channel is a small fraction of the width of the waterway. The design channel depth is frequently much greater than the natural depths of the area it traverses. The material beyond the channel limits is often a fine-grained sediment, unconsolidated and easily moved by the stronger currents or readily stirred into a suspension in shallow water by waves. A thin layer of sediment thus transported from outside of the channel into the channel can deposit many millions of cubic yards of material in the channel.

III-7. Navigation channels in tidal waterways are sometimes shoaled by sediments that are transported as littoral drift along the ocean shoreline adjacent to the mouth of the waterway. The material in littoral transport frequently spills into the estuary from both directions and accumulates in or adjacent to the entrance as inner and outer bars; it rarely moves far upstream if the material is sand, but it may move an appreciable distance if the material is silt or clay.

III-8. Organic material is derived from sewage and the remains of animal and plant life existing in or near the waterway. Appraisal of the importance of this source of sediment for the shoaling of tidal bodies has not been undertaken, but it is conceivable that it is highly significant in certain environments. It is to be remembered that limestone, dolomite, and certain other sedimentaries are the end results of forms of life, and great thicknesses of such rocks exist.

III-9. It has only recently been realized that improper deposition of spoil from channel dredging procedures may be a source of shoaling. However, when it is remembered that free dumping, agitation dredging, and disposal of pipeline dredge effluents into undiked disposal areas either do not remove the shoal material from the waterway or permit it to return to the waterway, it appears necessary to accept the premise that the dredged material is as much a potential source of shoaling material as is extraneous material dumped into the waterway by man, or virgin material entering the waterway naturally from the upland. Instances where the spoil is dumped year after year into the deep areas are known. In fact, this practice is still being carried on at present (1965). In most cases, the deeps do not retain the material as evidenced by the fact that they remain deep. It moves to the shoal areas, possibly back to those from which it was dredged.

III-10. A discussion of the sources of material causing shoaling of navigation channels in tidal bodies would be incomplete without considering their significance in solving the shoaling problem. Obviously, shoaling problems would not exist if sediments did not enter the waterway, or if they did not deposit therein but were flushed out to sea. Determination of the sources of the sediments in the shoals, followed by a critical examination of the possibilities of preventing their entrance to or eliminating them from the tidal body, constitute logical steps in solving shoaling problems.

III-11. Regrettably, it is rarely possible to establish the source of the sediment in the shoal by study of the shoal itself. The standard methods of examination include density and particle size determinations, also petrographic examinations. It is frequently found that all of the material in areas adjacent to the shoal and for considerable distances away from it is essentially the same as that in the shoal; therefore it is difficult to determine the source of the shoaling.

III-12. It is nevertheless desirable to investigate the characteristics of the shoal materials and all possible sources thereof, both immediate and ultimate. It is occasionally found that a source can be identified, and that something can be done to eliminate the supply or at least reduce its volume. In some cases, when it is believed that a source can be choked off but it cannot be identified by soil studies, the use of radioactive tracers may be justified as the means by which the source might be confirmed.

III-13. When such procedures fail, as is often likely to be the case, determination of the sources of shoals by deductive procedures becomes necessary. Such procedures include measurements of the sediment inflow from the upland, close examination of man's operations to determine whether he is introducing material into the estuary or into its upland tributary area, and study of changes in the hydrography of the waterway itself. It is apparent that scour in one locality produces sediment for shoaling in another. Study of dredging procedures is of course necessary in any event. Study of the hydraulics of the waterway will be necessary in such a deductive investigation to determine whether, or if so in what quantities, material entering the tidal waterway is flushed out to sea.

Mechanics of Transport and Deposition

III-14. The sediments reaching tidal waterways from all sources are transported by river and tidal currents and by density currents. Wave action is a factor at the entrance and in the shallow areas along the course of the waterway. The sediments are deposited where the currents no longer are competent to continue the transport of the sediments, or where the forces imposed by the ebb and flood currents are in balance. Saline intrusions modify the currents and, with the flocculation that they help produce, play a major role in causing depositions. The phenomena involved are very complicated, and are difficult to observe because of the huge size of most of the most important tidal bodies. As a result, the state of exact knowledge (i.e. based on observation and measurements) is deficient; reliance must be based upon general observations, data obtained from model experiments, and deductions.

III-15. Sediment brought into a tidal waterway by fluvial discharges may range in size from cobbles down to colloids. This material is transported seaward with decreasing efficacy as the distance downstream from the head of tide increases. It is usually found that the cobbles, gravel, and coarse sand deposit between the head of tide and the point where a flood current is first felt. Fine sand, silt, clay-size particles, and the colloids are moved back and forth with the flood and ebb currents. Most of the particles of silt size or coarser are deposited at every slack water or at some current of less than peak strength, depending on particle size. The fine particles are picked up when the succeeding flow attains sufficient strength. Some of the coarser particles remain in place until an unusually strong flow is experienced, due perhaps to a freshet or an abnormal tidal condition, or both. Thus, the upland sediment is sorted as to size and density from the head of tide to some point downstream, with the largest and heaviest particles settling out near the head of tide and most of the smallest and lightest particles depositing farther downstream.

III-16. If the tidal waterway has an irregular cross-sectional area, the tidal current cycles will vary from section to section. Where the current velocities are relatively low, permanent deposition of a larger portion of the load may be expected than where the currents are stronger. In fact, where the current velocities are relatively strong, perhaps no permanent deposition will occur. Thus, in most cases shoaling is experienced in certain reaches and not in others.

III-17. In tidal bodies that receive an appreciable upland discharge, certain effects caused by saline intrusions are experienced. The particulars as to these effects are discussed elsewhere in this report, and it is sufficient here to summarize the matter by pointing out that it is unlikely that sediment coarser than clay can pass to sea except during the most extraordinary conditions. This is due to the effect of the salinity intrusion on the distribution of currents in the vertical. Depending on the relation of the freshwater flow to the magnitude of the tidal prism, the intrusion may be a well-defined wedge or any intermediate form from well-defined to more or less completely mixed, in which case no salinity wedge can be detected. Except in the well-mixed estuary wherein the salinity from top to bottom is much the same, the water at and near the bottom is more saline, quite often much more so, than the water in the upper layers of the estuary. Seaward from a point between the advance and retreats of the salinity intrusion, the preponderant direction of flow in the lower layers may be upstream. Sediment in the waterway is flocculated by the sea water, and once it enters the lower strata it is trapped and is carried toward the upstream end of the salinity intrusion either to deposit in its course of travel or fluctuate with the tide, upstream and downstream across the section where there is neither upstream nor downstream preponderance, until it deposits as a shoal.

III-18. The location of the point in a tidal waterway (that receives upland, or fluvial discharges) where the salinity produces a hydraulic regimen, as described previously, varies. During low upland discharges, it is found farther upstream than when high upland discharges are experienced. During extraordinary floods, the entire waterway may be fresh water and the sediment load is discharged into the sea, possibly along with the shoals that have accumulated during low flows. Such events are rare, especially in those estuaries which have been deepened artificially for navigation. It is generally found that heavy shoaling occurs at the location in the estuary where the upstream predominance of flood flow at the bottom over ebb flow there is first encountered. In most cases, the suspended load received by the waterway from upland sources is deposited somewhere between the head of tide and this effective "barrier" to further movement of sediment seaward. There is no method known at present to cause flushing of the sediments to the sea, such as regularization of the waterway, whereby this condition can be circumvented. The only known method of combating the salinity barrier to seaward movement of sediment is to divert the fresh water from the waterway, thereby producing a system with water of homogeneous density from surface to bottom and eliminating bottom flood predominance; this diversion may also divert a major source of shoaling being introduced into the harbor.

III-19. It has been stated that material in the bed of the tidal waterway is a source of shoaling of navigation channels. This material may be scoured from one locality by excessive current velocities, and deposited in another as a result of slow currents, or because of the salinity intrusion discussed above. The material may move from shallow areas into the channel as a result of readjustment of the slopes of the bed of the waterway after the navigation channel is deepened. This process may take a long time to complete, and shoaling of the channel from this source may be heavy in the early years after the initial deepening of the channel. However, the shallow areas may continue to be the immediate source of navigation channel shoaling long after stability of the banks has been achieved. The shallow areas probably receive a large portion of the sediment introduced into the tidal waterway from all sources, including improperly conducted dredging operations, and much of this material is deposited there. Thereafter, it is resuspended by wave action, from time to time, and carried into the navigation channel by gradient, currents, and density flow.

III-20. Littoral drift has been listed above as one of the sources of sediments that shoal navigation channels in tidal waterways or in the entrances thereto. This is the material which is transported along the beach and nearshore areas by waves and currents. The sum of the drift volumes in each direction per unit of time is the gross littoral drift rate, whereas the difference is referred to as the net rate, the predominant direction being determined by the larger component. The net rate is the significant value concerning supply and demand requirements for stability of the shore or in estimating the years required to fill the impounding area updrift of a jetty, but the gross rate more accurately represents the shoaling potential in the entrance channel. It should be recognized, however, that the initial filling of the shadow zone of the jetty (i.e. that area protected from the refracted waves from the downdrift direction) will approximate the total rate of drift from the updrift direction; once this shadow zone has been filled, the subsequent filling will be at the net rate. In the case of a natural unimproved tidal inlet, littoral material is carried seaward by the ebb flow and deposited to form an offshore bar. Conversely, the flood current, assisted by wave action, carries material inward to similarly form an inner bar in the region where the current velocity lessens. The ebb and flow of tides maintain one or more channels through both bars; the dimensions of such channels depend primarily upon the volume of tidal flow. The intermittent deposition and scouring of littoral material cause these channels to shift more or less continuously. The abnormal tides or wave action accompanying storms periodically result in major displacement.

III-21. The mechanics whereby littoral material works its way across an inlet are not precisely known. It is known, however, that the predominant direction of littoral drift in the region very close to the shores in the immediate vicinity of an inlet is toward the inlet on either side. Thus, in the case of very large inlets such as Delaware Bay and Chesapeake Bay, the predominant drift is northward for a distance of several miles to the south of the inlet even though the predominant drift is southward throughout this general coastal region. It may be hypothesized that the gross drift rate is being introduced more or less continually at varying points along the channel, accumulating at slack water and being flushed out at the strengths of ebb and flood flow. Some material no doubt works its way to the downdrift shore by traversing the outer bar. Sudden shifting of the bar channel to an updrift position facilitates transfer of large volumes of material from the bar to the downdrift shore in a relatively short time, thus accounting for the customary instability of the shore downdrift from inlets.

III-22. The migration cycles of most inlets are very unpredictable. In areas where the drift rate is large and predominantly from one direction, the inlet may migrate in the direction of the drift movement for many years. The recycling breakthrough in these cases usually seems to be triggered by a storm surge of unusual intensity which overtops the barrier beach separating the elongated inlet from the beach. Where the drift rates are more evenly balanced, the migration cycles appear much more indefinite and the longitudinal extent of the migration seems much less than in the former case. The present understanding of inlet behavior is not sufficient, however, to establish a quantitative prediction of inlet migration, although historical records of the behavior of a specific inlet can be considered as a guide to the probable range of migration.

III-23. When a natural channel through an inner or outer bar is significantly deepened or widened by dredging alone, shoaling may occur at a rate approaching the gross littoral drift rate. In any event, it is likely that such deepening or widening will increase the shoaling rate of the inlet.

III-24. The customary method of stabilizing and improving outer bar channels is the construction of jetties flanking the channel. The primary purpose of jetties is to prevent littoral material from reaching the channel; however, in many localities their function as a breakwater to reduce wave action in the channel is of equal importance. The elimination of littoral drift as a shoaling source minimizes the problem of maintenance on the inner as well as the outer bar. In planning for the use of jetties, littoral drift characteristics must be carefully evaluated both in terms of the effects of updrift accretion and downdrift erosion. In an increasing number of cases it has become necessary to provide for mechanical bypassing of the littoral drift at inlets. The design of bypassing systems is still somewhat in the experimental stage but can be expected to develop rapidly during the next decade.

CHAPTER IV

SALINITY INTRUSIONS IN ESTUARIES

by

A. T. Ippen and G. B. Keulegan

General

IV-1. The effective use of tidal estuaries in the economic life of all nations calls for increased interference by engineering measures in the natural estuary environment. Channels are deepened and widened for navigation, the freshwater inflow is modified by control structures and estuary waters are polluted by waste disposal from adjacent industry and communities. These activities affect the natural conditions in three basic areas:

a. Tidal stages and tidal velocities are modified.

b. The local salinities are changed in magnitude.

c. The sediment deposition patterns along the estuary are changed with respect to location and intensity.

These consequences of man's interference are interrelated and means of predicting them must be found if the economics of improvements are to be evaluated properly.

IV-2. The area most readily susceptible to analysis is tidal dynamics. Tidal elevations and tidal velocities can be predicted for estuaries on the basis of theoretical treatments combined with measurements in models or in the field. Depending on the geometry of the estuary and on the tide-producing forces at the mouth, tidal flows can be determined usually for the entire estuary either by graphical or numerical integration methods or, for favorable geometries, by analytical functions. This topic is covered in Chapter II, and only the characteristics of importance to salinity intrusion are briefly reviewed here as follows:

a. The tidal wave length λ is sometimes of the same order as the length of the tidal reach of the estuary but usually exceeds the latter considerably. The geometric quantities involved in the analysis are related in the following order:

$$\text{wave length } \lambda > \text{channel width } b > \text{channel depth } h > \text{tidal amplitude } a$$

b. The rise and fall of the tide at the mouth is associated with a large transport of sea water into and out of the estuary. The total volume exchanged between high and low tide is known as the tidal prism which for a given estuary depends primarily on the tidal amplitude.

c. Tide-generated velocities are normally larger than the mean velocities necessary to transport the freshwater upland discharges to the sea. Also the volume of fresh water moving to sea in any one tidal period is usually smaller than the tidal prism. The mean convective velocity for the freshwater discharge thus is often of little consequence for the analysis of the tidal flows. The case of the stable salinity wedge, however, is an exception, as discussed later.

d. The same conclusion is reached with respect to convective currents generated within the estuary as a result of density differences due to salinity variations in the vertical and horizontal directions.

IV-3. In conclusion, the essential features of the tidal wave motion in estuaries can be determined from the theory for a homogeneous fluid. These results can then be applied to the analysis of salinity intrusion phenomena. In the case of the unmixed salinity wedge, they provide a description of the transient position of this wedge, and in the case of the partially mixed or well-mixed condition of the salinity intrusion, they not only give the translation of salinities throughout the tidal cycle but contribute also to the analysis of the mixing process itself.

IV-4. The analysis of salinity intrusion into tidal estuaries is closely linked to the prediction of turbulence generation and energy dissipation throughout the channel length. Local salinities are the result of the complex interaction of horizontal and vertical convection of salinity by the transient and turbulent tidal shear flows, of the convective currents generated by the density gradients, and of the seaward convective velocities resulting from the freshwater flow into the estuary. Depending on the relative strength of these currents, the characteristics of the salinity intrusions are usually described in terms of the observed salinity distribution as follows:

a. The unmixed or completely stratified case with a fairly well defined interface or discontinuity in salinity distribution.

b. The partially mixed state in which the local salinity varies vertically a large amount in terms of the local mean salinity.

c. The well-mixed condition in which the salinity variation over a vertical section varies only a small fraction from the local mean salinity.

It is readily apparent that cases a and b may be clearly distinguished while the difference between cases b and c is subject to a somewhat arbitrary definition of the dividing line. Cases a and b or b and c may occur in the same estuary under suitable conditions of tidal and freshwater flows.

IV-5. With ocean salinities and thus density differences confined within narrow limits, it is also seen that the above order of the various types of salinity intrusions encountered corresponds to one of increasing tidal effects and of decreasing freshwater convection currents. In this connection then the significance of the ratio of freshwater flow per tidal cycle to tidal prism becomes apparent with large ratios of the order of unity conforming to case a and with smaller ratios defining cases b and c. Examples are the Mississippi for the former and the Delaware estuary for the latter case.

IV-6. The theoretical and experimental approach to the prediction of the state of salinity intrusion in a given estuary has not as yet succeeded beyond the simple case of a rectangular channel of constant cross section.[1]* However, through these attempts, many aspects of general validity to actual problems have been clarified. In particular, the internal mechanism of the diffusion process and the related internal circulations are understood with respect to the corresponding variations of the salinity in the longitudinal and the vertical direction. These will be dealt with in detail subsequently. Case a will be discussed in a separate section since it represents a well-defined problem with distinct characteristics of its own and has been under study by Keulegan[2] as a limiting case in which the two fluids involved essentially maintain their identity without marked mixing.

IV-7. Sediment transport and deposition in estuaries with their engineering connotations provide one of the primary motives for the analysis of salinity intrusion in estuaries. Experimental and field studies have conclusively shown that the density currents engendered by the saline waters in estuaries and their interaction with the tidal flows are primarily responsible for the shoaling problems encountered. So far, the prediction of sediment movement from tidal and salinity intrusion studies must remain purely qualitative. The topic is dealt with on this basis in detail in Chapter III. It is mentioned here only to point out that sediment can affect the problem of salinity intrusion to the extent that long-range changes in channel geometry may occur, but it is not a factor in tidal and salinity intrusion analysis for given channels for a relatively short period of time.

Basic Elements of Salinity Intrusion

IV-8. The basic conditions producing the temporal mean current patterns observed for the zone of salinity intrusions in estuaries are illustrated in fig. IV-1. Cases a and c described in paragraph IV-4 are represented schematically. If the instantaneous velocities due to tide, freshwater flow, and density variations are averaged over a tidal cycle, the temporal mean velocities are obtained, to which this discussion is restricted in the following. The convective velocities responsible for the salinity intrusion may thus be considered in the steady state. The velocity patterns shown in fig. IV-1 represent therefore these

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* Raised numerals refer to similarly numbered items in Literature Cited at the end of this chapter.

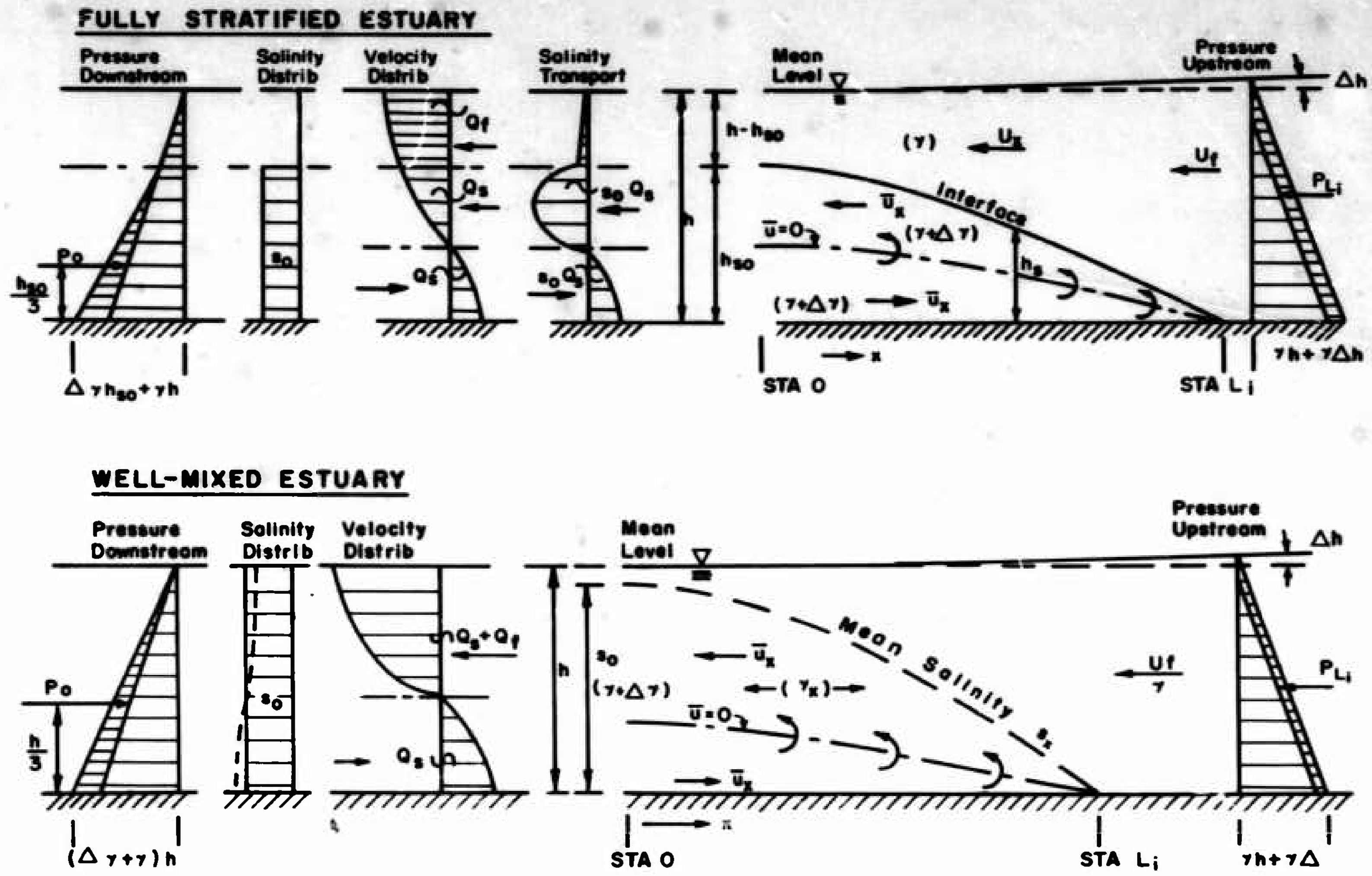

Fig. IV-1. Schematic representation of salinity intrusions in estuaries mean velocities generated by the salinity gradients and the freshwater flow. A flow of fresh water is assumed to enter at $x = L_i$* and to leave the channel at $x = 0$ at the saltwater basin. A saline water flow in the upstream direction underflows in both cases the seaward flow, which is essentially undiluted fresh water in the stratified condition and a mixture of fresh water and salt water in the mixed-flow case. Therefore, a line of zero longitudinal mean velocity must separate the upstream and downstream currents through which, however, vertical convection of saline water takes place into the upper layers flowing seaward. Both cases will be discussed subsequently in some detail.

IV-9. The salinity wedge (the unmixed state with full stratification) is characterized by the fact that essentially three layers of flow exist. Near the bottom a saltwater flow takes place in the upstream direction confined between the bottom and a line of zero horizontal velocity; above this line, the saltwater flow is reversed into the downstream or seaward direction; finally (above the distinct interface), the freshwater flow is seaward. The flows in the two saltwater layers are of decreasing magnitude toward the toe of the salinity wedge and must satisfy the continuity condition such that the vertical transfer of salt water through the line of zero horizontal velocity equals the rate of change of flow in each saltwater layer. The stratification requires that the continuity condition apply to both transport and fluid flow below the interface.

IV-10. The driving forces for the individual layers may be readily explained in principle by referring to the pressure diagrams. At station 0, the salinity produces an additional hydrostatic pressure linearly increasing in magnitude below the interface to a maximum intensity of $\Delta\gamma h_{so}$. At station L_i, the hydrostatic pressure force is increased by a uniform amount of $\gamma\Delta h$. The summation of these forces results in upstream gradients in the lowest layer of flow and in downstream gradients in the two upper layers. The local gradients at each depth are compensated by the rate of change of momentum of the local fluid masses and by the internal shear stresses. The equilibrium, or steady state, condition is seen, however, to be subject to a moment which affects the circulation experienced by the salt water. The resulting motion pattern is thus quite distinct from the normal channel flow.

IV-11. The stable salinity wedge with a well-defined interface exists due to the fact that the density difference alone results in currents below the interface which are too weak in turbulence generation to engender mixing. The interfacial waves also remain of minor consequence due to the stabilizing gravitational effects. Thus the salinity wedge in some estuaries, such as the Mississippi River

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* Symbols are defined in the Glossary at the end of this chapter.

mouth, may even be moved to and fro by the weak tides without breaking up. Such conditions of weak tides and large freshwater flow are characterized by a ratio of total freshwater transport per tidal cycle to tidal prism approaching unity, and thus this ratio may be employed with reservation as a parameter of classification. It is clear that the stable salinity wedge can only be broken up by introducing a relatively large turbulence-producing mechanism independent of the current system produced by the density differences themselves.

IV-12. For the partially and well-mixed states of salinity intrusion, the schematic representation in fig. IV-1 may again be referred to. The conditions of the mixed-flow estuary differ in various essential features from the stratified estuary. The intrusion is no longer clearly delineated by a physical near-discontinuity in salinity at an interface, but is now definable only by plotting mean values of salinities throughout the intrusion length. A finite salinity is measured locally as a time average at the free surface, and depending on the state of mixing, this average salinity will increase by a smaller or larger percentage toward the bottom. Where this increase is of the order of less than 50 percent of the surface salinity, the degree of diffusion might be arbitrarily termed "well mixed," and for variations higher than this the term "partially mixed" may be employed. Of significance for either of these conditions is the fact that because the state of mixing is usually the result of tidal action, the density flows which still are generated by the longitudinal and vertical gradients of salinity are now coupled to this external mixing process and cannot be defined separately.

IV-13. For the purpose of simplifying the more complex situation, the lower sketch in fig. IV-1 assumes essentially uniform mixing at the seaward end so that the hydrostatic pressures can be given there as linearly increasing as the result of the nearly uniform salinity distribution. The pressure at the bottom is thus increased by $\Delta\gamma h$. The total pressure at station 0 must be compensated by the hydrostatic pressure at station L_i, but again a moment is present which requires a rate of change of momentum of the fluid flowing and results in internal circulation, neglecting shear on the boundary for the present. In view of the smaller moment as compared to the stratified case, smaller velocities are to be expected for the internal circulation, assuming the mean salinities at the section to be the same. It is important, however, to note that this moment and hence this circulation must always be present as long as density difference exists between the fluids being mixed. The strength of the circulation is dependent on the density difference on one hand and on the magnitude of the freshwater flow on the other. The higher the freshwater flow, the more difficult it will be to maintain a nearly uniformly mixed condition. It is clear that for a given state of tidal action the salinity will tend from well mixed toward a partially mixed state as the freshwater flow is increased and the circulation currents become more intense within the shorter intrusion length. It is also concluded that there must always be salinity differences over the depth as long as fresh water moves through the estuary. Complete mixing will only be approached as the mean freshwater velocities assume negligible values either by low discharges or by widening of the section.

Analysis of the Stratified Estuary

IV-14. The theoretical and experimental developments of the salinity intrusion in the form of the saline wedge in the absence of strong tidal currents are primarily associated with the work of G. H. Keulegan,[2] and therefore the following sections represent essentially a condensation of his findings. In fig. IV-2, he and others[2,3,4] have shown that the behavior of density currents can be described best by means of a "densimetric Froude number" as follows:

$$F_d = \frac{U_f}{\sqrt{\frac{\Delta\rho}{\rho} g h}} \qquad \text{(IV-1)}$$

Moreover, it has been shown that the stability of a density current requires that this Froude number not exceed unity, since at this value interfacial waves will form, the interface will break up, and mixing will ensue.

IV-15. The existence of a stable salinity wedge requires therefore that the densimetric Froude number at the ocean end of the estuary have a value of unity or

$$F_{do} = 1 = \frac{U_f^o}{\sqrt{\frac{\Delta\rho}{\rho} g (h - h_{so})}} \qquad \text{(IV-2)}$$

Expressing this relation in terms of the Froude number at the upstream end rather than at the seaward end of the intrusion length by using from continuity:

$$U_f^o (h - h_{so}) = U_f h$$

the limiting height of the salinity wedge h_{so} may be expressed as:

$$\frac{h_{so}}{h} = 1 - (F_{d_{Li}})^{2/3} \qquad \text{(IV-3)}$$

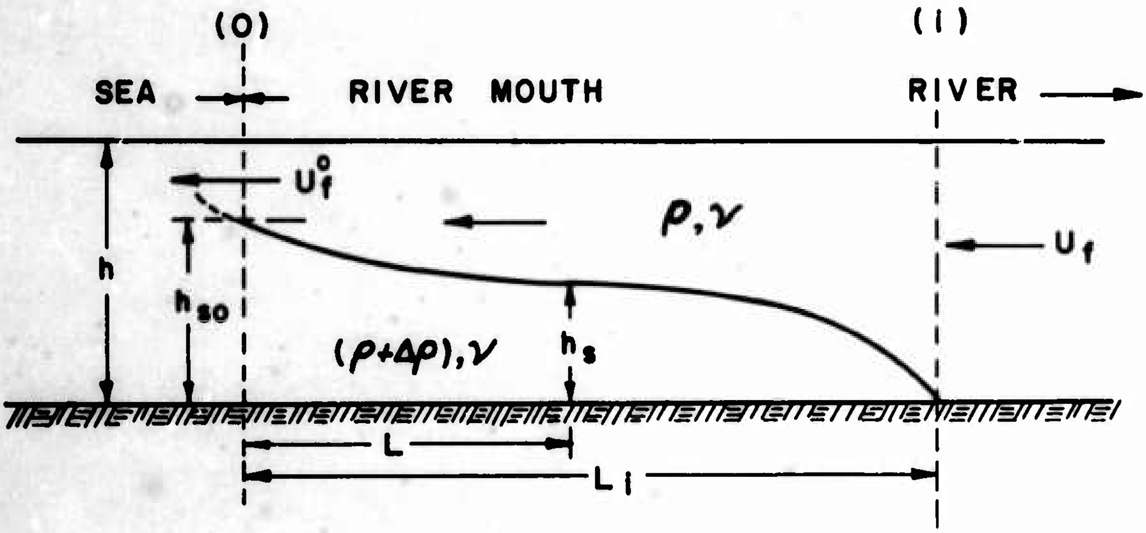


Fig. IV-2. Arrested saline wedge

with

$$F_{d_{Li}} = \frac{U_f}{\sqrt{(\Delta\rho/\rho)\,gh}} = \frac{U_f}{V_\Delta}$$

Keulegan has refined this result which permits the approximate computation of the saltwater depth at the river mouth from experimental measurements which are summarized in table IV-1.

Table IV-1

Depth of Saline Water at the Estuary Entrance

$F_{d_{Li}}$	h_{so}/h Experimental	$F_{d_{Li}}$	h_{so}/h Experimental	$F_{d_{Li}}$	h_{so}/h Experimental
0.05	0.815	0.25	0.555	0.45	0.402
0.10	0.718	0.30	0.518	0.50	0.375
0.15	0.660	0.35	0.480	0.60	0.310
0.20	0.608	0.40	0.438	0.75	0.232

IV-16. The length of the salinity wedge L_i has also been established by Keulegan for various laboratory conditions. On the basis of "an experimental evaluation of internal shear stresses and of their theoretical generalization," he was able to state the length of the saline wedge for channels of large width as:

$$\frac{L_i}{h_o} = 6.0 \left(\frac{V_\Delta h^{1/4}}{\nu}\right) (2F_{d_{Li}})^{-5/2} \qquad \text{(IV-4)}$$

$F_{d_{Li}}$ and V_Δ are defined with equation IV-3 and ν is the kinematic viscosity of water. It is specified that for this expression to be valid the densimetric Reynolds number $V_\Delta h/\nu$ must be greater than 10^7. For Reynolds numbers lower than this, other specific equations are given in the same reference and should be referred to when needed.

IV-17. The validity of the equation IV-4 was checked by Keulegan against some observations on the saline wedge of the Mississippi in South Pass where the intrusion length was measured as 14 miles for a discharge of 100,000 cfs. Assuming the depth as 45 ft, the width as 1500 ft, the temperature of the water as 20 C, and the density difference as $\Delta\rho/\rho = 0.02$, the freshwater velocity is $U_f = 1.48$ ft/sec and the value of $V_\Delta = \sqrt{(\Delta\rho/\rho)\,gh} = 5.38$ ft/sec. The substitution of the appropriate values in equation IV-4 gives a salinity wedge

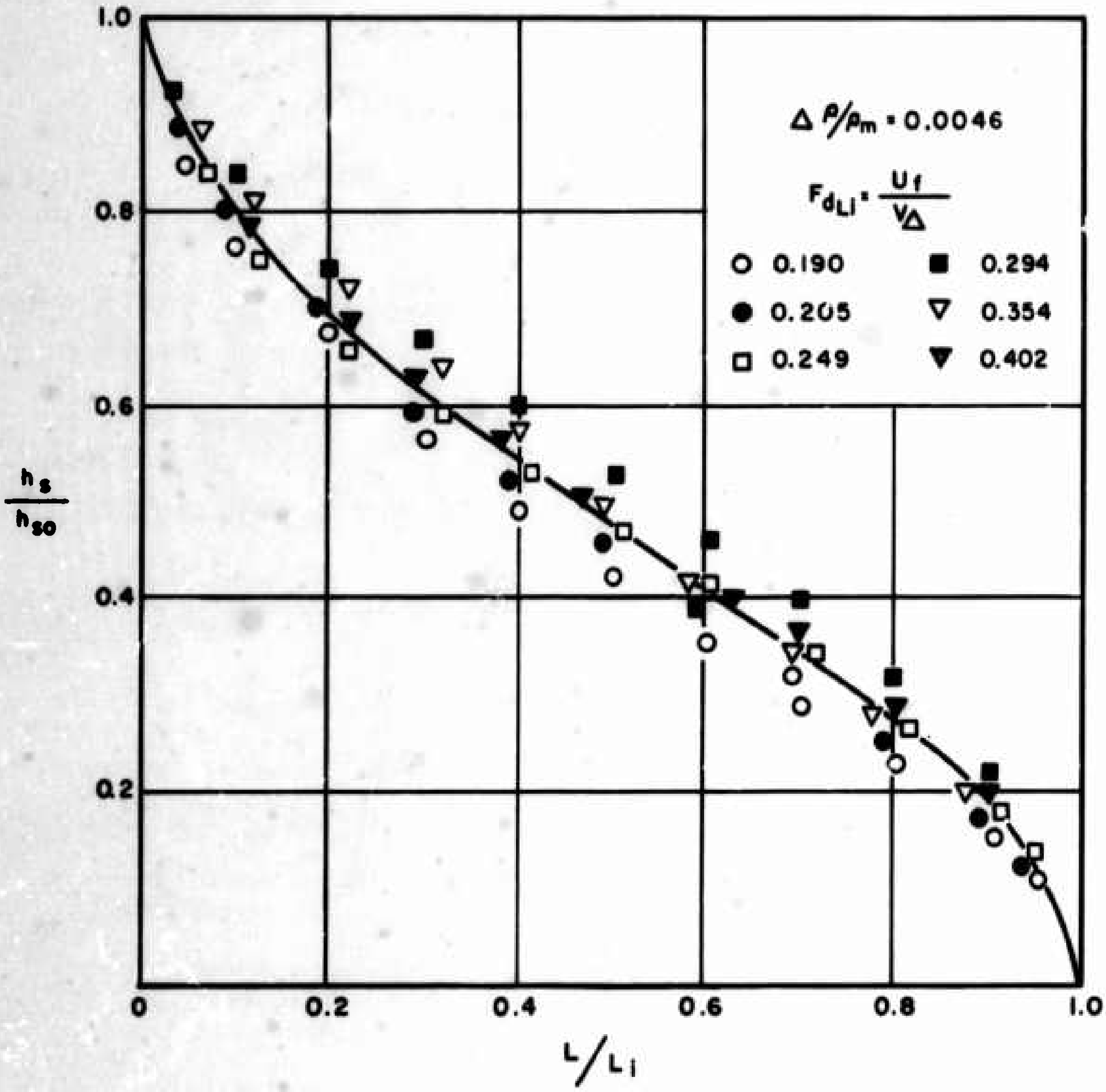


Fig. IV-3. Affine form of arrested saline wedges length of 15.5 miles, which shows satisfactory agreement with the observed value considering that only approximate data were available for the calculation.

IV-18. With the limiting depth h_{so} at the seaward end and the length of the intrusion L_i established, the shape of the entire saline wedge may be described in dimensionless form. Let the height of the wedge be h_s at any distance L from the seaward end, and the shape may then be given by the ratios h_s/h_{so} and L/L_i. Table IV-2 gives this affine shape and fig. IV-3 shows the experimental evidence for a set of runs with a density difference of $\Delta\rho/\rho = 0.0046$ and for various initial densimetric Froude numbers.

Table IV-2

Affine Shape of Arrested Saline Wedge

L/L_i	h_s/h_{so}	L/L_i	h_s/h_{so}	L/L_i	h_s/h_{so}	L/L_i	h_s/h_{so}
0.00	1.000	0.25	0.647	0.50	0.468	0.75	0.318
0.05	0.885	0.30	0.608	0.55	0.440	0.80	0.280
0.10	0.812	0.35	0.570	0.60	0.410	0.85	0.240
0.15	0.748	0.40	0.538	0.65	0.380	0.90	0.189
0.20	0.685	0.45	0.500	0.70	0.345	0.95	0.138
						1.00	0.000

Values in tables IV-1 and IV-2 are independent of viscosity and channel width and hold for values of ${F_d}_{Li}$ smaller than one-half.

The Vertically Mixed State of Salinity Intrusion

IV-19. The resistance encountered by the reversing current system created by stronger tides produces higher internal shear forces and a state of turbulent motion capable of overcoming the stabilizing effects of the density variation at the interface. Transfer of salinity takes place in the vertical direction and the fresh water flowing through the intrusion length becomes increasingly mixed with the saline ocean water. This state of salinity intrusion is characterized generally by finite salinity gradients in the three coordinate directions, x, y, and z, instead of discontinuities at an interface. The magnitude of the vertical gradients defines the state of mixing achieved more or less arbitrarily (as given in paragraph IV-12) as "well mixed" or "partially mixed." The internal circulation induced by the density differences is weakened (see paragraph IV-13), but remains as a powerful mechanism to augment the mixing in combination with the turbulent diffusion process. The dispersion of fresh water moving seaward through the intruding saline water in an estuary is thus subject to analysis only by considering both the "convective current" system engendered by the density differences and the "turbulent diffusion" process generated by the shear flows of tidal origin.

IV-20. The general differential equation describing this process is derived from the law of conservation of mass applied to the salt in this case and from the continuity condition for the liquid flow.[5] The process defining the state of salinity concentration s at a given point in terms of the mean velocity components u, v, and w in the x, y, and z coordinate system is given by:

$$\frac{\partial s}{\partial t} + u\frac{\partial s}{\partial x} + v\frac{\partial s}{\partial y} + w\frac{\partial s}{\partial z} = \frac{\partial}{\partial x}\left(D_x\frac{\partial s}{\partial x}\right) + \frac{\partial}{\partial y}\left(D_y\frac{\partial s}{\partial y}\right) + \frac{\partial}{\partial z}\left(D_z\frac{\partial s}{\partial z}\right) \qquad \text{(IV-5)}$$

The left-hand side of the equation represents the changes in s with time and through convection, which must be compensated by the turbulent diffusion-transfer process as defined on the right by salinity gradients and the diffusion coefficients D_x, D_y, and D_z. No general solution for practical cases is possible so far. However, a number of field and laboratory investigations have amply defined the general implications of the theory with respect to the consequences for actual estuaries. Some of these studies will be discussed in the following paragraphs.

IV-21. Since the convective process is generally important to the state of mixing in a given estuary, the various terms in equation IV-5 cannot be easily dismissed and must be carefully evaluated. Pritchard[6,7] has shown the relative importance of some of the terms from field studies on the James River, the Potomac River, and the Chesapeake Bay. Of interest here is that under certain conditions, especially in wide rivers, the transverse components (z direction) in equation IV-5 may be significant. Salinity differences on opposite banks of a wide estuary may be caused by convective currents induced by the Coriolis force. Transverse currents may also be generated by wind action. In a meandering estuary, transverse circulation during tidal flow may greatly augment the mixing process. However, no quantitative evaluation is possible at present by analytical means alone and the analytical approach has been confined essentially to the one-dimensional condition or to certain restricted two-dimensional problems.

IV-22. The first attempts to analyze salinity intrusions in estuaries on a one-dimensional basis were based on the freshwater-tidal prism ratio, and were concisely presented by Ketchum.[8] He divided the estuary into segments equal in length to the average tidal excursion of fluid particles during flood tide. Since complete mixing was assumed in each segment, this approach must be confined to well-mixed estuaries. It has been demonstrated lately[1] that the tidal prism ratio is not an adequate parameter to indicate similar mixing conditions, due to the fact that gravitational convective currents can almost never be neglected in the mixing process. The "mixing length" concept was next introduced by Arons and Stommel;[9] this concept led to the integration of the one-dimensional conservation-of-salt equation on the basis of certain assumptions for the horizontal eddy diffusion coefficient. Since, however, the length of the salinity intrusion must be known to define the salinity profile, this approach is useful primarily in the classification of estuaries and requires extensive field observations of salinity.

IV-23. Both of the above treatments result in the salinity distribution as averaged over a tidal cycle and do not give the time history of the distribution. Certain progress has been made since to include not only the variation of salinity with time over the intrusion length, but also to improve the prediction of the necessary diffusion parameters directly from an analysis of the tide-generated flow conditions.

IV-24. The analytical prediction of salinity intrusion, even for the one-dimensional case, is faced with the following complications:

- a. Tidal action produces a shear flow in the estuary that is variable with time and distance. Energy dissipation thus is also a function of time and distance.
- b. Density differences also are responsible for an internal flow pattern which varies with time and distance.
- c. The correlation of energy dissipation with salinity diffusion is dependent on the interaction of the shear flow generated by tidal action with the circulation produced by density differences.

IV-25. Any analytical approach, therefore, even on a one-dimensional basis must of necessity evolve from carefully planned experiments which permit the evaluation of the various assumptions necessary to solve the theoretical equations. This process is still active, but a number of solutions have become possible on this basis. Some of these are presented in review in the following paragraphs.

IV-26. For the one-dimensional case, equation IV-5 reduces to:

$$\frac{\partial s}{\partial t} + u \frac{\partial s}{\partial x} = \frac{\partial}{\partial x}\left(D_x \frac{\partial s}{\partial x}\right) \qquad \text{(IV-6)}$$

The quantities involved represent averages over short times as compared to tidal times, and it is further assumed that the salinity at any section x can be represented by its mean value at the section. Since u is the time dependent velocity of convection, it is replaced by the difference of the tide-generated velocity $u_{(x,t)}$ and the freshwater velocity $U_f = Q_f/A$; hence, $u = u_{(x,t)} - U_f$. Since, for a steady freshwater flow at the same tidal action, the salinity at a given station must recur at corresponding tidal times, the equation may be split into two parts, one dealing with the translation of the salinity with the horizontal tide and the other representing the restoration of the salinity by the diffusion process made necessary by the mean displacement downstream by freshwater flow. Hence:

$$\frac{\partial s}{\partial t} + u_{(x,t)} \frac{\partial s}{\partial x} = 0 \qquad \text{(IV-7a)}$$

$$U_f \frac{\partial s}{\partial x} + \frac{\partial}{\partial x}\left(D_x \frac{\partial s}{\partial x}\right) = 0 \qquad \text{(IV-7b)}$$

Equation IV-7a may be referred to as the salt-transport equation and equation IV-7b as the salt-diffusion equation.

IV-27. The salt-transport equation has been solved[1] observing that within the range of normal salinity intrusion maximum local tide velocities may be approximated by a linear decrease from the maximum tidal velocity u_o at the estuary entrance. Harmonic functions are assumed for the variation with time and the horizontal tides, i.e. the velocities at the estuary entrance are related to the vertical tides, i.e. the amplitudes at the seaward end from the superposition of damped progressive waves entering and leaving the estuary. For example, it has been shown that the tidal behavior in an experimental channel[1] as well as in two actual estuaries[10,11] may be approximated satisfactorily throughout the tidal length on the basis of damped cooscillating tides. With the above assumptions the equation IV-7a may be integrated for a channel of uniform section into the functional form:

$$\frac{s}{s_o} = f\left[\left(1 - \frac{a}{h}\frac{\sigma}{u_o}x\right) e^{-\frac{a}{h}\cos \sigma t}\right] \qquad \text{(IV-8)}$$

This equation describes the salinity variation with distance x and time t in terms of the amplitude to depth ratio a/h, of the tidal frequency $\sigma = 2\pi/T$, and of the maximum tidal velocity u_o at the seaward end. In order to utilize equation IV-8, however, the salinity distribution must be known at a given tidal time. The distribution desired can be obtained from the solution of the salt-diffusion equation IV-7b.

IV-28. The one-dimensional form of the salt-diffusion equation was obtained by ignoring the vertical and lateral mass transfer and diffusion terms in equation IV-5. While this is necessary at the present state of knowledge, it is clear that the presence of the density differences within the intrusion length gives rise to internal mass transfer and circulation greatly augmenting the turbulent mixing expressed by the turbulent diffusion coefficient D_x as originally defined. The internal circulation generated by density differences represents an eddy motion with a scale many times larger than the normal eddy scale associated with turbulent shear flow. The diffusion coefficient D_x must therefore be replaced by an "apparent diffusion coefficient" D'_x understood to express the enhanced ability of the system to diffuse as a result of the gravitational convection currents.

IV-29. For the estuary of constant section and thus of constant freshwater velocity, equation IV-7b can be readily integrated to:

$$U_f s = - D'_x \frac{\partial s}{\partial x} \qquad \text{(IV-9)}$$

The further solution depends on information regarding the variation of D'_x with x. Two such solutions have been carried out and tested,[5,1] the first assuming a constant value of $D'_x = D'$ and the second introducing D'_x as decreasing inversely with x from an initial value D'_o at the estuary entrance at x = 0. Thus:

$$\text{for } D'_x = \text{constant} = D' : \frac{s}{s_o} = e^{-\frac{U_f(x+B)}{D'}} \qquad \text{(IV-10)}$$

$$\text{for } D'_x = D'_o \frac{B}{x+B} : \frac{s}{s_o} = e^{-\frac{U_f}{D'_o}\frac{(x+B)^2}{2B}} \qquad \text{(IV-11)}$$

The integration constants were determined in both cases from the boundary condition $s = s_o$ for $x = -B$. It is readily seen that at the entrance (x = 0) the ocean salinity s_o cannot be maintained at all times, but that the salinity s here must be lower than s_o for a considerable period during ebb flow, in order that the appropriate freshwater amount may be passed into the ocean. A distance B is therefore introduced as that hypothetical length of channel extending into the sea to $x = -B$ where the salinity at all tidal times is maintained at ocean strength s_o.

IV-30. For the evaluation of the quantities B and D'_o, observations of the salinity variation with time at the ocean entrance only are required. B is then given by equation IV-8 by introducing the time interval from low water to the time when ocean salinity is reached at the ocean entrance. With B known and introducing also the measured minimum salinity at the entrance, the apparent diffusion coefficient D_o' may be determined from equation IV-11. The distribution of the mean salinity s may then be computed for all values of x and for all tidal times by combining equations IV-8 and IV-11. This one-dimensional procedure has proved satisfactory for the evaluation of experimental results obtained in a uniform channel as described in reference 1.

IV-31. By defining the maximum intrusion length L_i arbitrarily as extending to the section in the estuary at which the mean salinity reaches a maximum value of only 1 percent of the ocean salinity, equations IV-8 and IV-11 may again be used to calculate the length L_i as shown in the same reference.

IV-32. In the preceding summary of analytical developments, the dependence of the salinity distribution on tidal action and freshwater flow has been demonstrated. It is desirable, however, to link the diffusion coefficient, defined so far only by experimental observations of the salinity at the entrance, directly to the energy dissipation produced by the tide-generated shear flow. The Kolmogoroff theorem as shown by Batchelor[12] gives the diffusion coefficient D for homogeneous turbulence in terms of the dissipation and of the mean size of the turbulent eddies. It has been shown that by analogy this relation may be adapted for correlation also for the case where diffusion is subject to gravitational convection due to density differences.[5] Thus, the apparent diffusion coefficient may be expressed by:

$$D_x' = N \ell'^{4/3} G^{1/3} \qquad \text{(IV-12)}$$

The rate of energy dissipation per unit mass of fluid is expressed as G; ℓ' is normally a linear measure of the scale of turbulence, but in view of the effect of gravitational convection currents included in D_x' it assumes much larger dimensions for this application; N is a constant. Generally, D_x' is variable along the estuary, but so far correlations have only been made on the basis of D_o'.[1,5] The rate of energy dissipation G must be determined from the tidal behavior and has been derived for certain cases[1,11] from a harmonic analysis of tidal waves for the entire estuary. Thus, for a given estuary for which the scale ℓ' of the turbulent mixing may be assumed as approximately constant, equation IV-12 furnishes the ratio of $D_o'/G^{1/3}$ as a significant parameter for analysis of diffusion.

IV-33. The effect of density variations and of freshwater flow must be considered also in relation to the energy dissipation by tidal action. Fresh water flowing through the intrusion length at the rate of $Q_f = bhU_f$ will be mixed with salt water and will gain in specific weight by an amount $\Delta\gamma$ or in potential energy by $\Delta\gamma h$. Thus, there exists a flux of potential energy per unit mass of

$\Delta\gamma/\gamma\, g\, h/t_r$ from the intrusion length seaward, wherein $t_r = L_i/U_f$ represents the average time of retention in the estuary. Hence, the parameter to describe the average rate of increase of potential energy per unit mass of fresh water passing through the estuary is:

$$J = \frac{g\,\Delta\gamma\, h\, U_f}{\gamma\, L_i} \qquad \text{(IV-13)}$$

It has been shown[1,5] that the ratio of energy dissipation by turbulence per unit mass expressed by G to this average gain in potential energy J per unit mass is most useful in correlating diffusion coefficients. Two independent experimental studies[1,5] have given convincing evidence of this, and their final results are summarized in the following paragraphs.

Experimental Studies of the Mixed-State Salinity Intrusion

IV-34. The study conducted at Massachusetts Institute of Technology and reported in reference 5 deals with the flow of fresh water through a rectangular channel toward a saltwater basin under mixing conditions produced by a set of screens extending horizontally over the entire length of the channel and being oscillated vertically to produce a uniform field of turbulence all along the channel. The degree of turbulence generated is dependent on the mesh size of the screens, and the amplitude δ and the frequency $\sigma_s = 2\pi/T_s$ of oscillation. The power dissipated per unit mass in the channel is constant for a given test condition in contrast to a channel subject to tides, where the dissipation of energy and the state of turbulence depend on the local shear field generated by the tidal flows. Thus, equation IV-10 expresses the salinity distribution to be expected. Fig. IV-4 gives the values of the apparent diffusion coefficient D' for many runs at different density differences and freshwater velocities plotted against the product $\delta\,\sigma_s$ which is proportional to the one-third power of the energy dissipation G. A wide variation in D' results for given values of $\delta\,\sigma_s$ which is governed by the density difference and the freshwater velocity. The values of D' reach a minimum when the density differences approach zero. The sloping line thus represents the condition of pure turbulent diffusion, since the convective density currents augmenting the diffusion process are no longer present and the diffusion coefficient is variable only with the one-third power of the dissipation G. Hence, the apparent diffusion coefficient D' equals D along this line of minimum values of D'.

IV-35. In fig. IV-5 these same data are presented again. The apparent

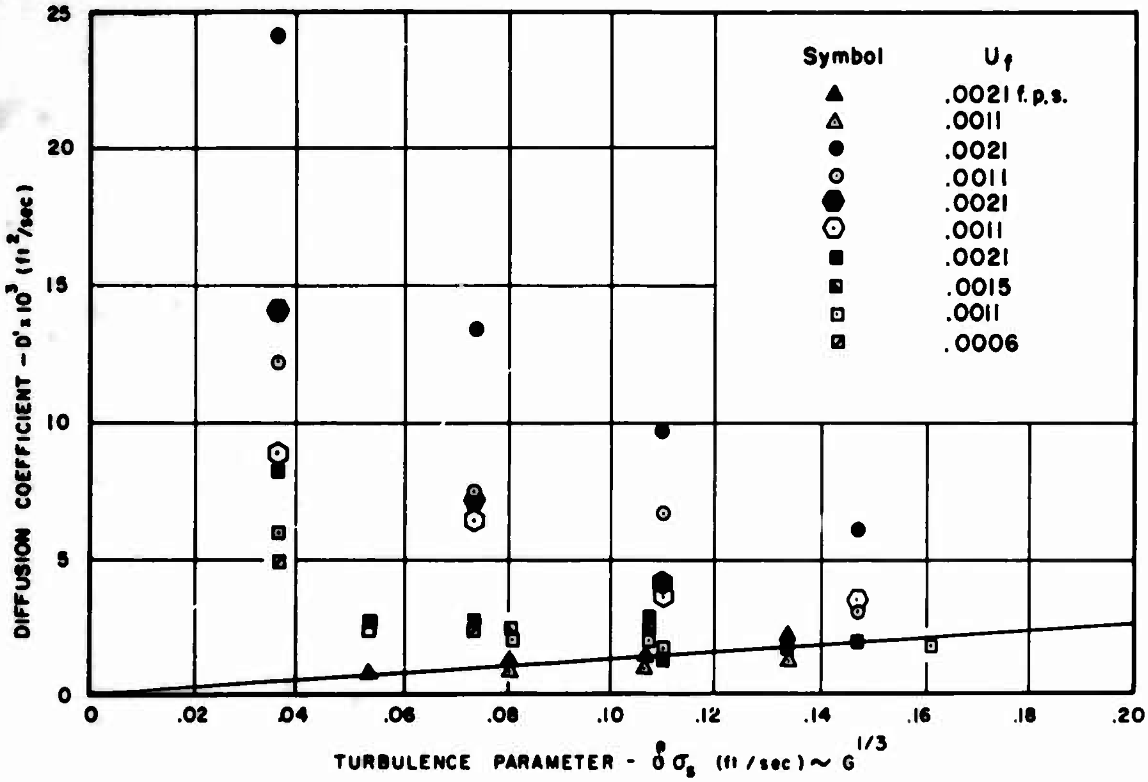


Fig. IV-4. Effect of freshwater flow and density difference on apparent diffusion coefficient

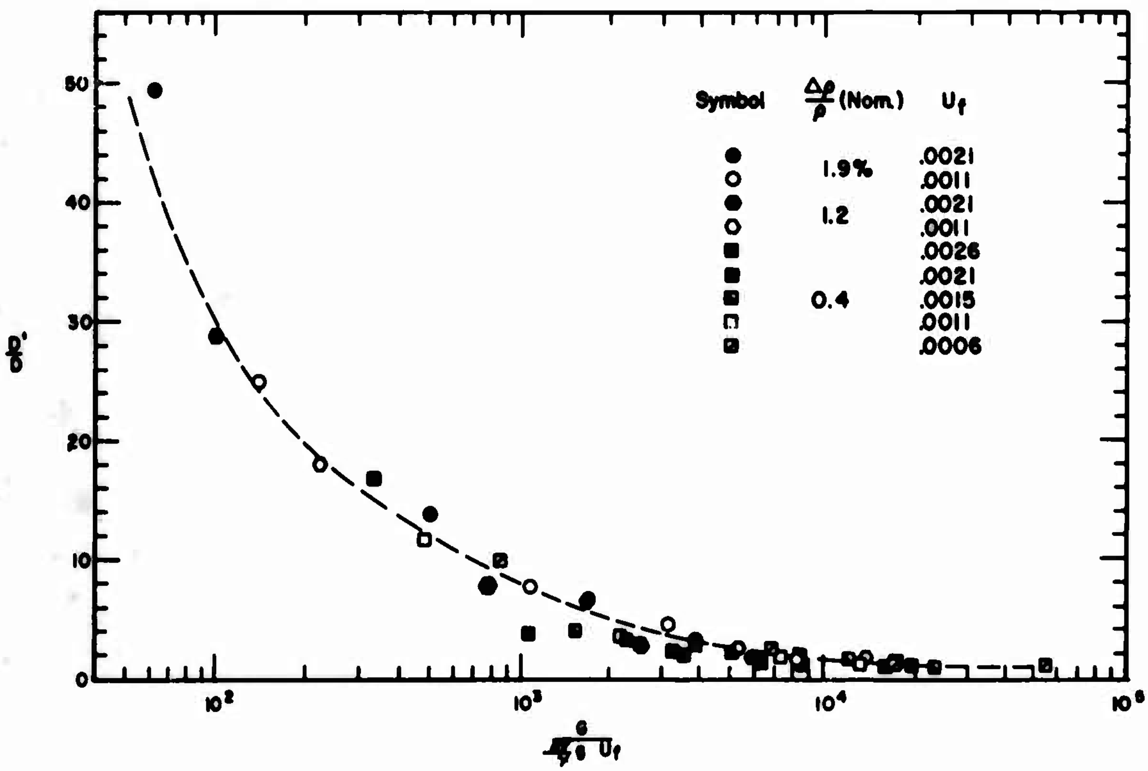


Fig. IV-5. Correlation of ratio of apparent and turbulent diffusion coefficient with stratification parameter

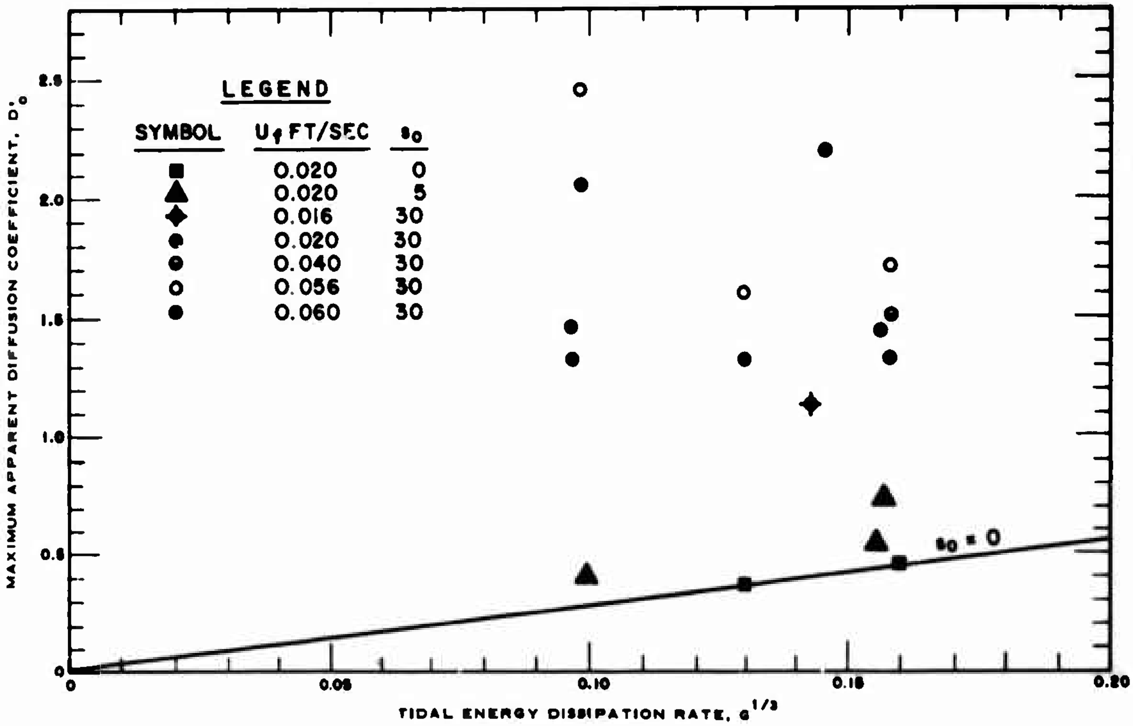


Fig. IV-6. Diffusion coefficient versus rate of tidal energy dissipation

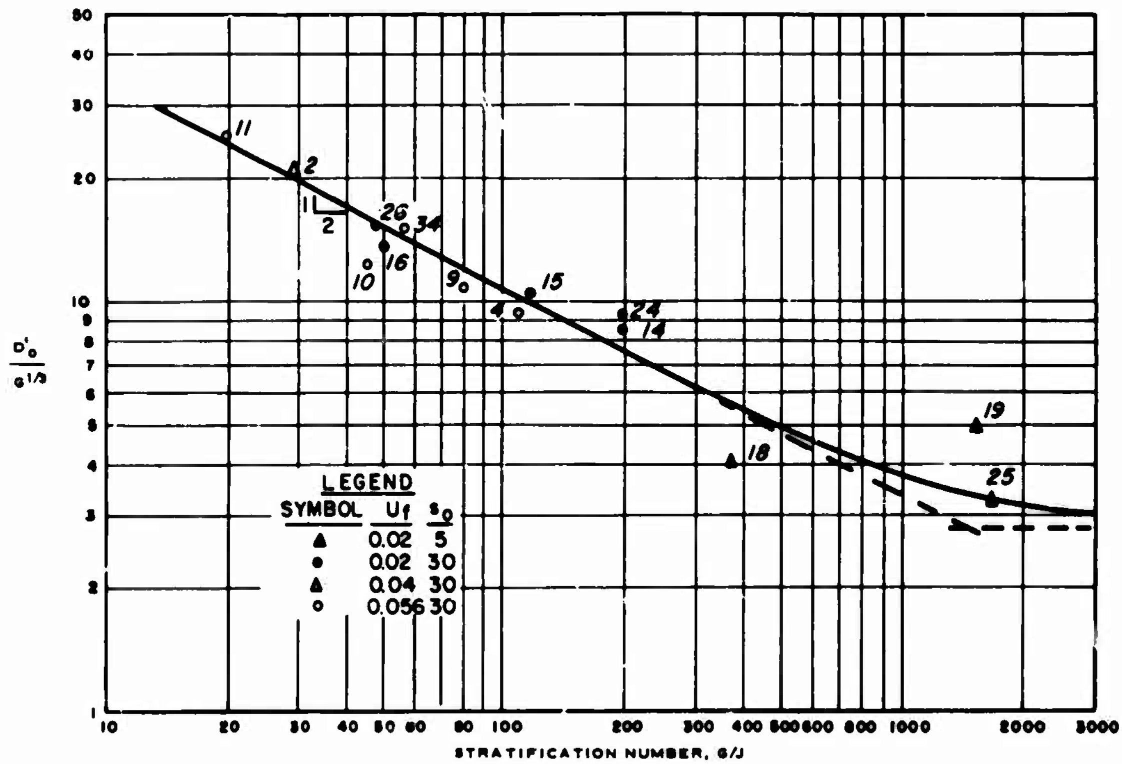


Fig. IV-7. Correlation of apparent diffusion coefficient D_o' with stratification number diffusion coefficient D' is given in terms of the pure turbulent coefficient D, thus expressing the effect of the convective gravity currents on the diffusion process while the energy dissipation G per unit of fluid mass has been referred to the rate of gain of potential energy per unit mass of the throughflow. In the latter ratio G/J, the quantities depth h and length L_i have been omitted (see equation IV-13) since they were essentially constant for all runs. The introduction of the stratification parameter G/J has clearly resulted in an effective correlation of the diffusion coefficients D' for all conditions of density difference and of freshwater flow.

IV-36. In fig. IV-6 the results of diffusion experiments conducted at U. S. Army Engineer Waterways Experiment Station[1] are shown analogous to fig. IV-4. In these experiments the mixing process, however, was produced by the resistance on the channel boundaries encountered by tidal currents in a long rectangular channel. The resulting turbulence, producing the diffusion of fresh water and salt water, is therefore neither steady nor constant along the channel. The apparent diffusion coefficients D_o' were evaluated therefore under the assumptions inherent in equation IV-11 for different density conditions and freshwater flows. The dissipation G per unit mass was derived from tidal calculations involving amplitude and period of tide[1] as well as the channel dimensions. Minimum values of $D_o' = D_o$ are again obtained for $\Delta\gamma/\gamma = 0$ or $s_o = 0$ (ocean salinity). The increase in D_o' for given tidal dissipation rates G is quite marked as freshwater flows are increased, indicating a strong trend toward more stratified flow conditions.

IV-37. Satisfactory correlation for all runs is again achieved by replotting the data analogous now to fig. IV-5 in spite of the one-dimensional character of the basic analysis. Fig. IV-7 presents the maximum apparent diffusion coefficient D_o' in terms of the one-third power of G in accordance with equation IV-12 in relation to the stratification parameter G/J, wherein J is defined by equation IV-13. It is seen that in agreement with the trends in fig. IV-5 high freshwater flows and larger density differences result in increased stratification; i.e. vertical salinity differences will increase. Larger tides in a given channel will give lower values of D_o', as will lower freshwater flows. In the limit a constant value of D_o' is approached for pure turbulent diffusion when vertical salinity differences tend to disappear.

IV-38. It has been claimed that the ratio of the volume of freshwater flow over a tidal cycle to the tidal prism, i.e. the volume of sea water entering the estuary during flood tide, may be employed as an index of the mixing characteristics of the estuary. It has been shown[1] that this tidal prism parameter has only very limited application for a given estuary and only with respect to freshwater flow. However, as tidal amplitudes, depths, and channel roughness are varied from one estuary to another, equal tidal prism ratios cannot be used to draw conclusions with respect to the state of mixing. The ratio G/J, i.e. the stratification parameter, however, will be a dependable index of the degree to which salinity varies over the vertical direction, no matter what the values of the basic variables involved. It thus also must be considered as a similarity parameter to be satisfied in simulating salinity intrusions in model estuaries.

IV-39. Finally, the one-dimensional results achieved so far by analysis and verified by experiment may be illustrated by fig. IV-8 showing the mean vertical salinities along a tidal channel for various tidal times throughout a tidal cycle. The solid lines have been calculated in accordance with the developments given in paragraphs IV-27 through IV-30. It is concluded that for channels of relatively uniform section the mean salinity conditions can be represented satisfactorily on the basis of the one-dimensional analysis. An extension of the work toward channels of more complex geometry is highly desirable; however, the basic variables governing the problem of salinity intrusion have been clearly identified and will not change. A qualitative extrapolation of trends in salinity intrusion phenomena is therefore possible for given estuaries at present on the above basis even if the channel shapes exhibit other configurations.

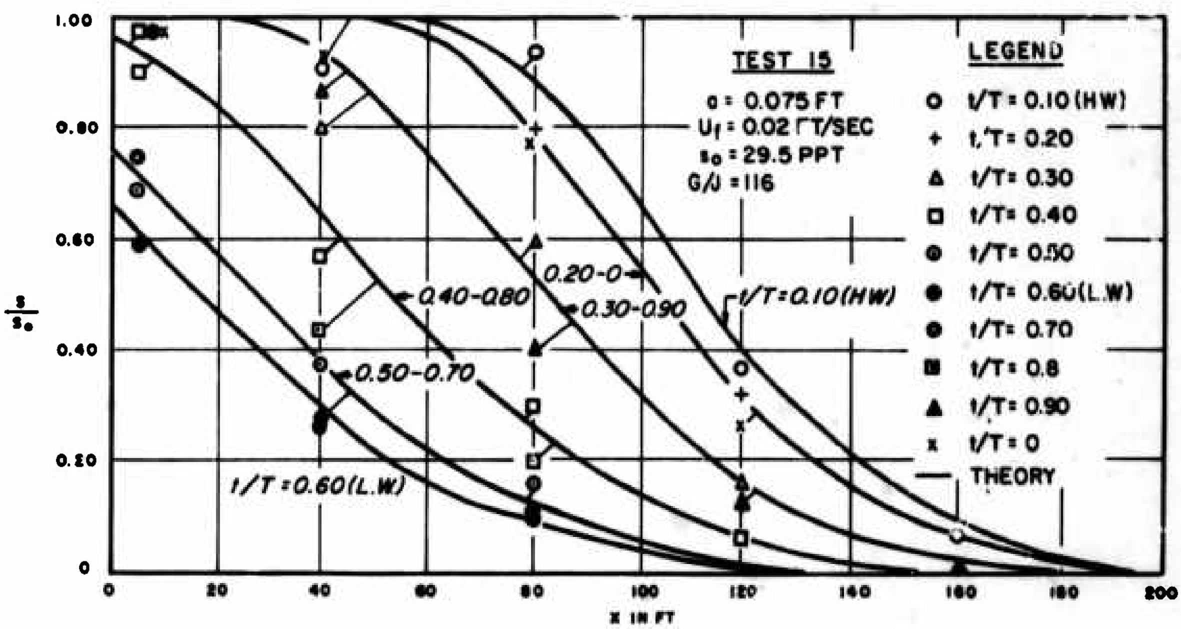
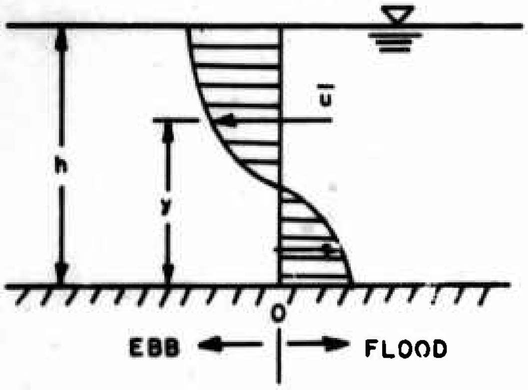
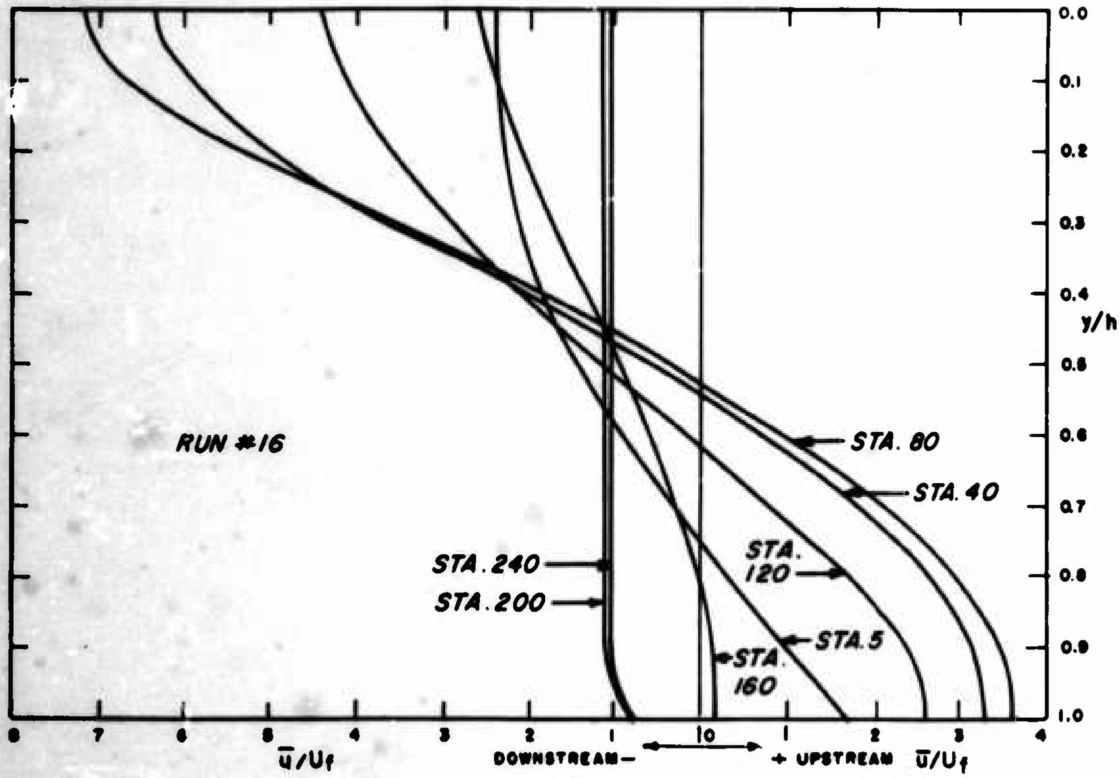


Fig. IV-8. Comparison of experimental and analytical instantaneous salinity distributions for a tidal cycle



a. VERTICAL VELOCITY DISTRIBUTION



b. HORIZONTAL VELOCITY DISTRIBUTION

Fig. IV-9. Time average, horizontal and vertical velocity distribution

Effect of Salinity Variations on Mean Velocity Distributions

IV-40. One of the most important aspects of salinity distributions in estuaries is the effect of the internal convective currents induced by density variations on the sediment transport. In reference to fig. IV-1, the general pattern of the circulation has already been discussed. In order to determine the strength of this circulation at various stations along the estuary, a two-dimensional solution of the general diffusion equation must be attempted. It is assumed that a steady state of the secondary current system is achieved by considering only the mean velocities $\bar{u}$ at any vertical section x remaining after averaging the instantaneous tidal velocities over a tidal cycle. In stratified estuaries, the mean velocity distributions obtained from experimental studies and verified by field measurements (see flow-predominance curves discussed in other chapters) exhibit the form illustrated in fig. IV-9a. It is clear that the integration of this curve must result in the net freshwater flow Q_f toward the sea; i.e.

$$Q_f = U_f \cdot A = \int_0^h \bar{u} b \, dy$$

This fact is used as a check on the time-averaging process for the instantaneous velocities.

IV-41. The mean velocity distribution curves for various stations along the estuary may next be compared by superimposing the curves as shown in fig. IV-9b, wherein the variables have been reduced to dimensionless quantities. Station 5 indicates a location 5 ft upstream from the ocean end of the experimental channel and station 200 is upstream of the end of the salinity intrusion. The mean velocities $\bar{u}$ are divided by the mean freshwater velocity U_f and the distance y is divided by the local depth h.

IV-42. It is found that the mean velocities $\bar{u}_b$ near the channel bottom vary along the channel from zero near the upstream maximum intrusion point to a maximum at some intermediate station and then revert to lower values near the ocean end of the channel as illustrated in fig. IV-10. These trends are the result of the effects of the longitudinal gradients of the salinity. In fig. IV-8, it is clear that $\bar{u}_b$ is weak near the limit of the salinity intrusion since the salinity gradients are small, that stronger bottom currents must exist at the intermediate stations since salinity gradients are highest, and that the bottom currents near the ocean end must decrease, since for a large part of the tidal cycle the salinity becomes equal to the ocean salinity and thus the average salinity gradients are small here also.

IV-43. It is also seen in fig. IV-10 that this variation of the upstream bottom velocities from weak to strong and back to zero along the salinity intrusion profile is accompanied by corresponding changes of the velocities over the entire section, the mean velocities toward the surface becoming stronger in the seaward direction in the zone of maximum salinity gradients. It follows from continuity that vertical downward velocities must exist near the ocean end and upward velocities near the intrusion end. An internal circulation pattern due to salinity variations is thus revealed in addition to the general seaward flow and the tide-generated turbulence which was previously discussed as instrumental in greatly enhancing the mixing process. To be noted also is the fact that the temporal mean velocities $\overline{u}$ exceed in intensity by far the mean freshwater velocities U_f.

IV-44. The two-dimensional mean flow patterns discussed previously have been obtained from experiments conducted following the two studies[1,5] in channels of uniform rectangular section. It must therefore be assumed that the mean current intensities in channels of variable section will differ from those illustrated, but qualitatively the circulation patterns will not be altered and the internal mechanism of the mixing process remains essentially the same with the effects on shoaling as discussed elsewhere. Further studies in this area should eventually result in correlating the mean circulation intensities with the parameters defined throughout this chapter as representative of the tidal dynamics,

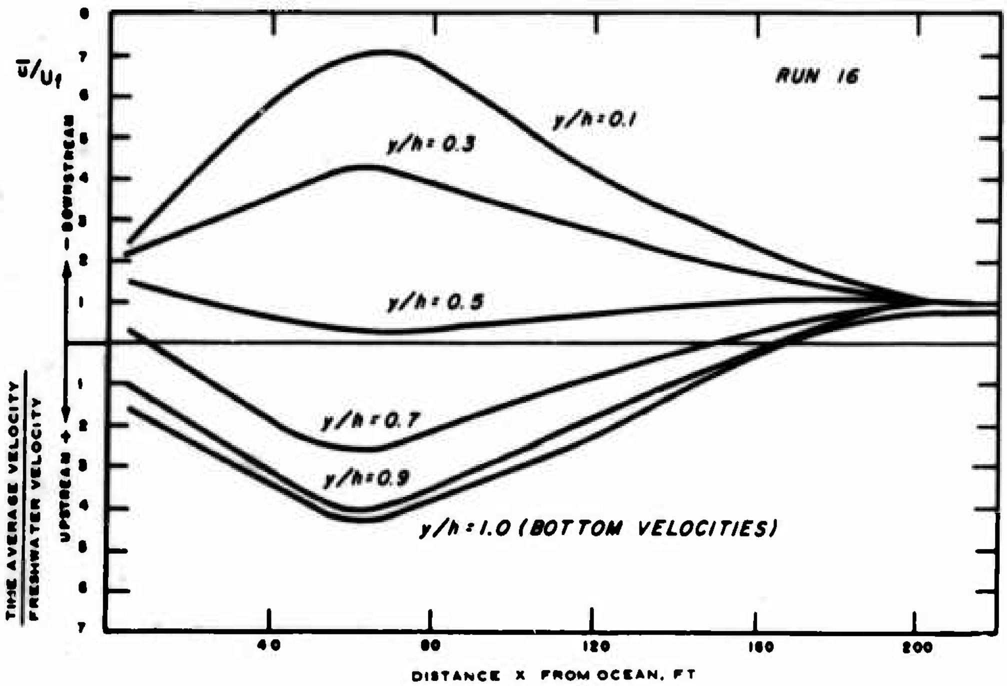


Fig. IV-10. Mean velocities along channel at various depths the density currents, and the freshwater flow in estuaries.

Considerations in Similarity

IV-45. To examine salinity intrusion in an estuary model (the upstream portion of which is free of salinity effects and the downstream portion of which is the main seat of salinity intrusion), both portions need to have flow similarities with the prototype. As the flow pattern in the upstream portion is hardly influenced by what transpires in the lower portion in regard to density details, then it suffices to obtain the similarity conditions for the upstream portion by imagining that the entire estuary is free of salt. The distortion is allowable. This may be verified from the fact that the dynamic actions are described with sufficient fidelity by a single differential equation involving the mean velocity averaged in a cross section. The usual assumptions entered into are: (a) the vertical accelerations are negligible and the pressures are hydrostatic, (b) the surface disturbances are small in comparison with depth, (c) the velocities in a cross section are practically uniform, (d) the channels are practically prismatic, and (e) the dissipations are due to a turbulence instigated by bottom roughness and leading to a quadratic resistance law. In addition to the dynamic equation, one needs to consider the condition of continuity best expressed by the Dupuit relation.

IV-46. Since the hydrodynamic phenomena can be described by differential equations, the latter are sufficient to determine the conditions for similarity between equivalent systems. For the purpose, the equations need be put in dimensionless form. In that manner, one arrives at transfer parameters, some of which give the rules to pass from the quantities of one system to the corresponding quantities of the other system, while the remaining ones specify what the similarity conditions are.[13]

IV-47. Analysis shows that when similarity exists and u and η are the mean velocities and the surface displacements, respectively, observed at corresponding stations x_p and x_m and at corresponding times t_p and t_m, then

$$\left(\frac{u}{\sqrt{gh_o}}\right)_m = \left(\frac{u}{\sqrt{gh_o}}\right)_p \qquad \text{(IV-14)}$$

and

$$\left(\frac{\eta}{h_o}\right)_m = \left(\frac{\eta}{h_o}\right)_p$$

where h_o is a characteristic depth. It is convenient to identify the latter with the mean depth of the cross section at the estuary mouth at the time that the sea is at its mean level. The corresponding times and distances are determined from

$$\left(\frac{t}{T}\right)_m = \left(\frac{t}{T}\right)_p$$

and

$$\left(\frac{x}{\lambda}\right)_m = \left(\frac{x}{\lambda}\right)_p$$

where T is the characteristic time and λ the characteristic length. These characteristic quantities are not independent of each other but are related by the equations

$$\left[\frac{T(gh_o)^{1/2}}{\lambda}\right]_m = \left[\frac{T(gh_o)^{1/2}}{\lambda}\right]_p = 1 \qquad \text{(IV-15)}$$

For estuaries, T may be identified with the tidal period. For model studies of floods in a river, T may be taken to represent the duration of a significant flood mode. It is to be noted that for an estuary model λ is not a geometrical element of the channel; its value is derived from equation IV-15 after h_o and T are specified. In this definition, λ would be the wave length of translation waves of period T moving with a celerity of $\sqrt{gh_o}$ in a hypothetical channel of constant depth h_o.

IV-48. In order that there be similarity between model and prototype, one must impose

$$\left(\frac{i\lambda}{h_o}\right)_m = \left(\frac{i\lambda}{h_o}\right)_p$$

where i is the channel slope. As the model channel will be constructed having its rigid boundaries affine with the prototype, the last relation is automatically satisfied. Obviously, distortion is allowed. A second condition for similarity is

$$\frac{i_m}{i_p} = \frac{f_m}{f_p}$$

where f is the coefficient of resistance of the rigid surfaces. If one forsakes geometrical affineness between the rugosities of model and prototype, the roughness of the model may be fixed by trials to accord with the preceding requirement. That affineness is not necessary results from the fact that expressing f in terms of τ

$$\left(\frac{\tau}{\rho u^2}\right)_m i_p = \left(\frac{\tau}{\rho u^2}\right)_p i_m$$

or using Reynolds equivalents involving the turbulent velocity fluctuations u' and v'

$$\left(\frac{\langle u'v'\rangle}{u^2}\right)_m i_p = \left(\frac{\langle u'v'\rangle}{u^2}\right)_p i_m$$

Accordingly, any kind of resistance element that can produce in the model the required fluctuations would be regarded as satisfactory. Note that if there is distortion of channel shape, the intensity of the velocity fluctuations in terms of mean flow is greater in the model than in the prototype.

IV-49. The above results show that the scales are Froudian. For example, denoting the scale for horizontal lengths as ℓ, that is $x_m = \ell x_p$, and the scale for depths as d, that is $h_m = d \cdot h_p$, the discharge to be introduced in the model (from equation IV-14) is

$$\frac{Q_m}{Q_p} = \ell d^{3/2}$$

and the corresponding times (from equation IV-15) are

$$\frac{t_m}{t_p} = \ell d^{-1/2}$$

IV-50. The Coriolis effects are to be ignored. Now, the transfer parameter for such effects is

$$\pi_\omega = \frac{\omega b}{\sqrt{gh}}$$

where b is the width of estuary channel, h is the mean depth of water in a cross section, and ω is the angular velocity of the plane containing the estuary.

As shown by Lord Kelvin, the crest lines of waves proceeding in the channel will be tilted instead of being horizontal, owing to the Coriolis effect. This may be ignored if π_ω is a small fraction negligible with respect to unity. Otherwise, for similarity

$$\omega_m = \ell^{-1} d^{1/2} \omega_p$$

and hence it would be necessary to place the model on a rotating table, which would be obviously difficult, if not impossible, with large models. However, this has been done with models reproducing currents in large bodies of water, such as Lake Michigan, and tidal portions of the sea.

IV-51. A consequence of distorting the model is the relative deepening of channel cross section. This would tend to modify the pattern of secondary currents in a plane normal to the main flow. In general, the nature of these currents is not well understood and the modification produced by the distortion cannot be appraised. Somewhat related to these phenomena are the effects of channel meanderings and bends. It will be necessary to suppose that these come with small curvatures and in a manner such that deploying the estuary model along a straight axis will hardly affect the accuracy of results. This notion is of value in the theoretical determination of pollutant motion.[14]

IV-52. In the earlier investigations of the U. S. Army Engineer Waterways Experiment Station on problems of saltwater intrusion, attention was given to the question of salinity scale. It was found solely on the basis of trials that if discharges are scaled according to Froudian rules, then the proper scale for salinity is unity. It is desirable to discuss briefly the limitations of the finding.

IV-53. Consider first the movement of saline fronts against a river current from the viewpoint of dimensionless analysis. If x_o is the distance traveled by the saline front during time t in a rectangular channel connected to the sea, and if the water depth is h, then

$$\frac{x_o}{h} = f\left(\frac{U_f}{V_\Delta}, \frac{V_\Delta h}{\nu}, \frac{V_\Delta t}{h}, \frac{h}{b}\right)$$

where U_f is the river velocity, V_Δ the densimetric velocity, ν the kinematic viscosity, and b the channel width. The densimetric velocity is given by $\rho V_\Delta^2 = \Delta\rho g h$, where $\Delta\rho$ is the excess of density of sea water over the fresh water ρ. As the validity of the above relation was very amply established in the National Bureau of Standards experiments, it is obvious that in a laboratory channel of horizontal bottom the intrusion phenomena may be studied with any density of sea water. The salinity scale is arbitrary. For a river model, denoting the slope of the riverbed by i, the corresponding general equation would be

$$\frac{x_o}{h} = f\left(\frac{U_f}{V_\Delta}, \frac{V_\Delta h}{\nu}, \frac{V_\Delta t}{h}, i\right)$$

where h is the depth of the sea at the river mouth. Assume that the model is not distorted. Hence, when

$$\left(\frac{U_f}{V_\Delta}\right)_m = \left(\frac{U_f}{V_\Delta}\right)_p$$

and

$$\left(\frac{V_\Delta t}{h}\right)_m = \left(\frac{V_\Delta t}{h}\right)_p$$

then

$$\left(\frac{x_o}{h}\right)_m = \left(\frac{x_o}{h}\right)_p$$

provided that the incongruities resulting from Reynolds number may be ignored. If the river velocities are to be determined on the basis of Froudian scales, a further condition is that

$$\left(\frac{U_f}{\sqrt{gh}}\right)_m = \left(\frac{U_f}{\sqrt{gh}}\right)_p$$

Thus

$$\left(\frac{\Delta\rho}{\rho}\right)_m = \left(\frac{\Delta\rho}{\rho}\right)_p$$

i.e. the salinity scale should be unity. The general expression is simplified to

$$\frac{x_o}{h} = f\left(\frac{U_f}{gh}, \frac{U_f h}{\nu}, \frac{U_f t}{h}, i\right)$$

This equation in its present form can neither confirm nor deny the possibility of distortion. However, the method of differential equations, as shown elsewhere,[15] would suggest that distortion is permitted provided that the interfacial stress between fresh water and the salt wedge is of similar nature in model and prototype. The difficulty may be avoided if the interfacial stresses in model and prototype are not large, and any incongruency or imbalance may be corrected by altering the roughness of the model in the area of intrusion. It would be sufficient, it appears, to have the total resistive forces on the salt wedge to correspond in model and prototype. This points, of course, to the necessity of model verification also in the problems of saline fronts or salt wedges.

IV-54. When there is complete mixing between fresh water and salt water, interface difficulties no longer exist and similarity arguments from differential equations show that distortion is valid, and if discharge input is determined from Froudian scales, then the salinity scale needs to be taken as unity.[13]

IV-55. In the elucidations made above, the effect of viscosity was ignored. This effect is always present in small models with smooth boundaries. A good illustration of this is the motion of a saline front originating from a tideless sea and entering into the still waters of a smooth channel. In the initial stages, the irrotational character of flow is conserved, and therefore the saline front heights and advance velocities are the same. Subsequently, there is continual reduction in height of front and velocity of advance because of dilution at the front from mixing and of interfacial resistance from viscous origin. Experiments show that if V_o is the initial velocity of the front and V the velocity after a travel distance L, then

$$\frac{V_\Delta}{V} = \frac{V_\Delta}{V_o} + \beta \left(\frac{V_\Delta h}{\nu}\right)^{-1} \frac{L}{h}$$

$$V_o = 0.57\, V_\Delta, \quad V_\Delta^2 = \frac{\Delta\rho}{\rho} \cdot gh$$

where β is a constant, the value of which depends on the depth-width ratio.[16] Obviously, this shows the very marked influence of viscosity in the later stages of motion.

IV-56. Another illustration would be a small smooth-walled lock model communication with an internal navigation channel. Let L_o denote the lock length and h_o the depth. The lock is filled with a liquid of density $\rho + \Delta\rho$ whereas the density of the channel water is ρ. Let V_o be the initial velocity of the saline front and V the velocity after a distance L is traversed by the front. Experiments show that

$$\frac{V}{V_o} = f\left(\frac{L}{L_o}, \frac{V_o h_o^2}{\nu L_o}\right), \; V_o = 0.462\, V_\Delta, \; V_\Delta^2 = \frac{\Delta\rho}{\rho} g h_o$$

In the earlier studies by O'Brien and Cherno, the term containing viscosity was implied. Later, in the National Bureau of Standards studies, the validity of the term was amply demonstrated.[16] For similarity

$$\left(\frac{V h_o^2}{\nu L_o}\right)_p = \left(\frac{V h_o^2}{\nu L_o}\right)_m$$

and as $\nu_p = \nu_m$ in actual practice, then

$$\frac{\left(\frac{\Delta\rho}{\rho}\right)_m}{\left(\frac{\Delta\rho}{\rho}\right)_p} = \frac{\left(\frac{h_o^5}{L_o^2}\right)_p}{\left(\frac{h_o^5}{L_o^2}\right)_m}$$

Accordingly, if in the model the lock shape is distorted, the salinity scale is other than unity, the exact ratio depending on the degree of distortion.

IV-57. In estuary models as ordinarily constructed, fortunately, the effect of viscosity may be ignored. The prototype flows are of turbulent character, and as the models will be distorted, by necessity, turbulence needs to be augmented. In models simulating tidal action, a second source of turbulence is the temporal unsteadiness of the tidal currents.

IV-58. In the absence of mixing as previously mentioned, two differential equations determine the conditions for similarity between prototype and model. To repeat, these are the dynamic equation of motion expressed in terms of mean velocity of flow and the Dupuit relation of continuity. When mixing is present, either partial or complete, similarity conditions are to be sought by considering the equation of turbulent diffusion. The case for complete mixing is simpler to treat. Let s be the temporal mean of the averaged value of salinity in a cross section at x. Denoting s/s_o by σ and x/L by ξ, where $L = T(gh_o)^{1/2}$, equation IV-7b may be written also as

$$\frac{d\sigma}{d\xi} = \frac{D_o}{u_o L}\frac{d}{d\xi}\left(\frac{D_x}{D_o}\frac{d\sigma}{d\xi}\right) \qquad \text{(IV-16)}$$

This suggests that for similarity in regard to salinity in the intrusion zone, not only must the value of D_o/u_oL be the same both in the model and in the prototype, but also the functional dependence of D_x/D_o on ξ must remain unaltered. The meaning of the statement is theoretical since one does not know, a priori, what the bearings of the hydrodynamic details such as tidal period, tidal range in the estuary mouth, sea salinity, river current, channel shape, and channel roughness are on the magnitude of D_o at the estuary mouth and D_x in the intrusion area. Even if one were enlightened enough to pinpoint the condition for D_o, the uncertainty of D_x/D_o shall persist, as this function takes on different courses in varying channels and estuaries.

IV-59. Because of this lack of a complete theory underlying the causes for the diffusion, the only course open to establish similarity between prototype and model is the method of trials. In the distorted model, selected on the Froudian scales and adopting unity salinity scale for the sea waters, the following quantities are made to correspond to the prototype data: (a) height fluctuations in amplitudes and with phases, (b) pattern of currents in the normal cross section, (c) salinity patterns in the normal cross section, (d) surface salinities for specified times, and (e) the extent of saline intrusion. The only means to establish the desired correspondence is the adoption of the necessary roughness in shape and distribution, and this is sufficient to cause the proper agencies for diffusion, the shear flow, the circulation pattern caused by density variation, and the temporal changes of tidal currents to correspond. The significance of the quantities G and J for D_o was previously explained. The affected correspondence of tidal elements implies that G and J also are corresponding.

IV-60. To say that the model corresponds to the prototype in regard to salinity implies that the ratio L_i/L where L_i is the intrusion length has the same value as in the prototype, and also that $s/s_o = f(x/L)$ has its value conserved. Accordingly, the deduction from equation IV-16 would be that

$$\left(\frac{D_o}{u_oL}\right)_m = \left(\frac{D_o}{u_oL}\right)_p$$

and

$$\left(\frac{D_x}{D_o}\right)_m = \left(\frac{D_x}{D_o}\right)_p$$

In other words, the prevailing diffusions of the prototype may be deduced from the diffusions observed in the model. This is a very beautiful and useful result, for the advance of a pollutant pulse in an estuary may be computed once the saline intrusion in the model is made to correspond to that in the prototype. The practical agreements between the studies by the U. S. Army Engineer Waterways Experiment Station and the theoretical work of Pritchard in this field show these findings to be true. That such transfers are possible is the result of the assumption that for a pollutant not changing the density field of a saline intrusion, the diffusion coefficients for salt and pollutant are the same. A pollutant particle in a portion of salt environment travels with the environment.

Glossary

Lower case letters

a	Tidal amplitude, ft
b	Channel width, ft
d	Scale for depths, $d = \frac{h_m}{h_p}$
e	Base of natural logarithms, 2.718 . . . .
f	Coefficient of resistance of rigid surfaces
g	Acceleration due to gravity, ft/sec^2
h	Mean depth of water in cross section, ft
h_o	Characteristic depth, i.e. mean depth of water in cross section at the estuary mouth at the time sea is at its mean level, ft
h_s	Local height of salinity wedge at $x = L$, ft
h_{so}	Maximum height of salinity wedge at ocean entrance, ft
i	Channel slope, ft/ft
ℓ	Scale for horizontal lengths, $\ell = \frac{x_m}{x_p}$
ℓ'	Mean eddy size, ft
m	Model
p	Prototype
s	Temporal mean salinity at any point s_o = ocean salinity or reference salinity
t	Time, sec
t_r	$\frac{L_i}{U_f}$ = time of retention of fresh water within intrusion length L_i, sec
u	Current velocity in x direction, ft/sec
u_o	Maximum tidal current velocity at estuary entrance, ft/sec
$\bar{u}_b$	Mean current velocity near bottom, ft/sec
$\bar{u}$	Temporal mean current velocity component in x direction, ft/sec
u', v'	Turbulent current velocity fluctuations, ft/sec
$u_{(x,t)}$	Tidal current velocity in x direction, variable with time and distance, ft/sec
v	Current velocity in y direction, ft/sec

w	Current velocity in z direction, ft/sec
	Horizontal distance, ft
x_p, x_m	Horizontal distances (model and prototype), ft
y	Vertical distance, ft
z	Transverse distance, ft

Upper case letters

A	Cross-sectional area of the estuary, ft^2
B	Hypothetical distance from estuary entrance to location of ocean salinity at all tidal times, ft
D	Diffusion coefficient without density differences, ft^2/sec
D'	Apparent diffusion coefficient including effect of density-generated convection currents, ft^2/sec
D_o	Maximum diffusion coefficient at estuary entrance without density differences, ft^2/sec
D'_o	Maximum value of D'_x at estuary entrance $(x = 0)$ including effect of density-generated convection currents, ft^2/sec
F_d	Densimetric Froude number
G	Rate of energy dissipation per unit mass, ft^2/sec^3
J	Rate of potential energy gain per unit mass of fresh water due to mixing with saline water, ft^2/sec^3
L	Distance upstream from estuary entrance, ft
L_i	Salinity intrusion distance, ft
N	Constant
Q	Discharge, ft^3/sec
Q_f	Total freshwater flow, ft^3/sec
Q_s	Saline water flow, ft^3/sec
T	Tidal period, sec
T_s	Period of screen oscillation, sec
U_f	Mean freshwater current velocity toward ocean, ft/sec
U_f^o	Mean freshwater current velocity at estuary entrance, ft/sec
V	Velocity of propagation of saline front, ft/sec

V_o	Initial velocity of propagation of saline front, ft/sec
V_Δ	Densimetric velocity, ft/sec

Greek letters

β	Constant, the value depending on the depth-width ratio
γ	Specific weight, lb/ft^3
δ	Amplitude of vertical oscillation of screens, ft
η	Surface displacement, ft
λ	Wave length of translation waves of period T moving in a channel of constant depth h_o
ν	Kinematic viscosity, lb sec^2/ft^4
π	3.1416
π_ω	Coriolis number
ρ	Density of liquid, lb sec^2/ft^4
σ	$\frac{2\pi}{T}$ = tidal frequency, sec^{-1}
σ_s	$\frac{2\pi}{T_s}$ = frequency of screen oscillation, sec^{-1}
τ	Shear stress, lb/ft^2
ω	Angular speed of rotation of earth multiplied by sine latitude, radians/sec
Δ	Finite interval symbol

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CHAPTER V
EFFECTS OF DENSITY DIFFERENCES ON ESTUARINE HYDRAULICS

by

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V-1. The hydraulic regimen of an estuary is due to the interactions of the tides, the winds, the geometry of the waterway, discharges of fresh water into the estuary, and salinity intrusions. The resultant currents transport sediments from sources to locations where, for various reasons, the currents are no longer competent to keep these materials in motion. One of the causes of this may be salinity intrusions. This chapter examines the mechanics of the interaction of salinity intrusions on the hydraulics of various estuaries, presents methods of analysis of the phenomena involved, and discusses the problems that are created. Chapter IV goes into the causes of salinity intrusions and their variability; this chapter discusses their effects.

V-2. The presence in estuaries of water of different and varying density causes marked differences in the magnitudes, distributions, and durations of the currents as compared with those of a system having water of homogeneous density from surface to bottom and throughout its length and width. As a result of the density differences between the heavier salt water at the seaward end of the estuary and the fresh water at the upstream end, each type of water tends to assume distributions that approximate the shape of a wedge with the base of the wedge at the source. The interface of (or line of demarcation between) the salt water and fresh water may vary from well-defined to obscure, depending on the degree of mixing of the salt water and fresh water in any given estuary. Where the mixing is slight, the transition from salt water to fresh water is well defined and occurs within a small percentage of the channel depth. On the other hand, where the mixing is appreciable, no definite interface of the salt water and fresh water exists, except in isolated instances, and wedge-shaped distributions of the salt water and fresh water cannot be discerned.

V-3. Knowledge of the degree of mixing of salt water and fresh water in an estuary is highly important, as this appears to control certain important features of the hydraulic and shoaling characteristics of an estuary. For convenience, the degree of mixing may be seperated into three broad categories: highly stratified, partly mixed, and well mixed. However, the transition from one type of mixing to another is gradual instead of well defined. In subsequent paragraphs, the more important characteristics of the highly stratified, partly mixed, and well-mixed estuaries are illustrated through the use of current velocity and salinity data obtained in Southwest Pass of the Mississippi River, Charleston Harbor, Savannah Harbor, and the Delaware Estuary.

V-4. In all estuaries in which there is significant stratification of the salt water and fresh water, the magnitudes, durations, and directions of the currents above the interface are appreciably different from those below. Fig. V-1 shows schematically the distribution of current velocities and directions in a highly stratified estuary; Southwest Pass of the Mississippi River is an excellent example. Upstream from the tip of saltwater intrusion (section A, fig. V-1) the direction of flow throughout the entire depth is toward the sea at all times with a vertical velocity distribution similar to that of an upland stream. Farther downstream (section B, fig. V-1), the flow in the freshwater strata is still toward the sea, but the direction of flow in the underlying salt water is usually upstream at all times. Of course, these conditions are not found at the same locations in a given estuary throughout the range of tidal and upland discharge conditions. The wedge moves upstream and downstream in accordance with variations of these factors. The cause of the continuous upstream flow within the saltwater strata, even though the extent of saltwater intrusion may be stable, is the additional hydrostatic pressure produced by the salinity and by the constant erosion of the salt water from the interface caused by the outflowing fresh water, thereby maintaining stability of the intrusion length.

V-5. The lengths of the flow direction arrows in fig. V-1 indicate the relative magnitudes of currents throughout the highly stratified estuary. Maximum velocities in the freshwater strata occur near the surface and near the entrance to the estuary, while the maximum currents in the saltwater strata occur near the bottom and near the upstream limit of intrusion. Generally speaking, velocities in the fresh water decrease with distance from the surface while those in the salt water decrease with distance from the bottom, and a point of zero velocity is found within, or in the vicinity of, the interface.

V-6. Rapid shoaling usually occurs in the region of the saltwater intrusion tip in a highly stratified estuary, as shown in fig. V-1. The heavier particles of sediment, which are rolled or pushed along the riverbed by the outflowing fresh water, come to rest as soon as the tip of intrusion is reached. The lighter particles, which are transported largely in suspension in the fresh water, gradually fall through the interface along the length of intrusion and are transported upstream by the saltwater currents to the vicinity of the tip, unless local conditions are such as to cause deposition before the vicinity of the tip is reached. The channel reach occupied by the upstream limit of intrusion is therefore a focal point for accumulation of sediment from both upstream and downstream, with the heavier particles depositing just upstream from the

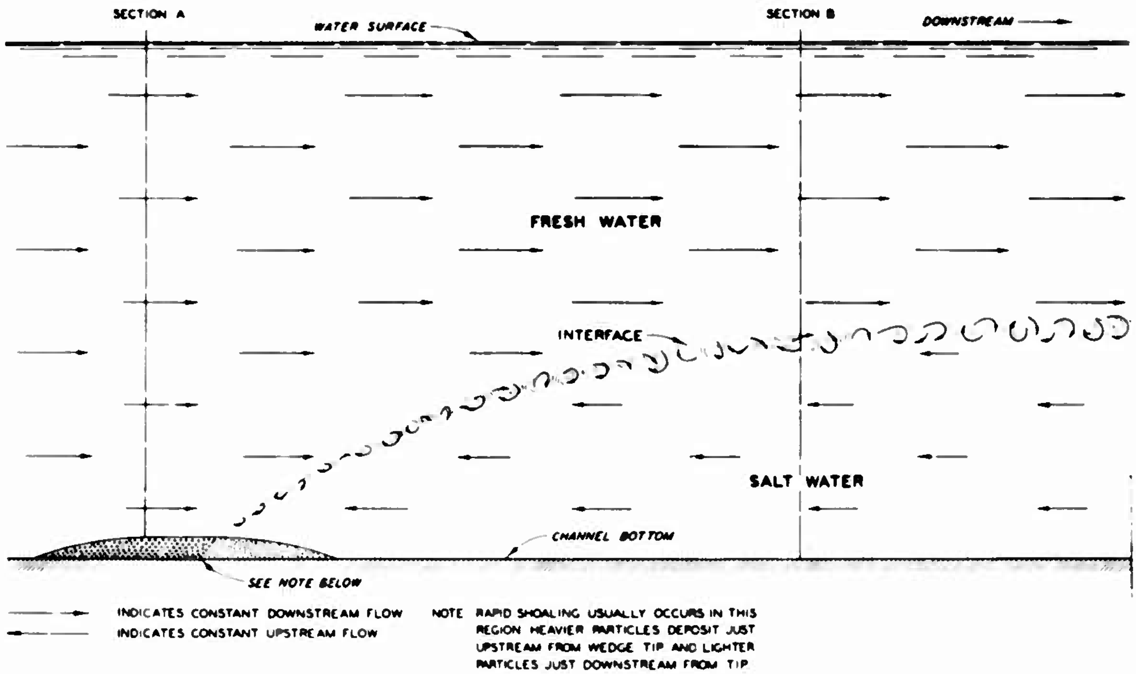


Fig. V-1. Conditions typical of highly stratified estuary

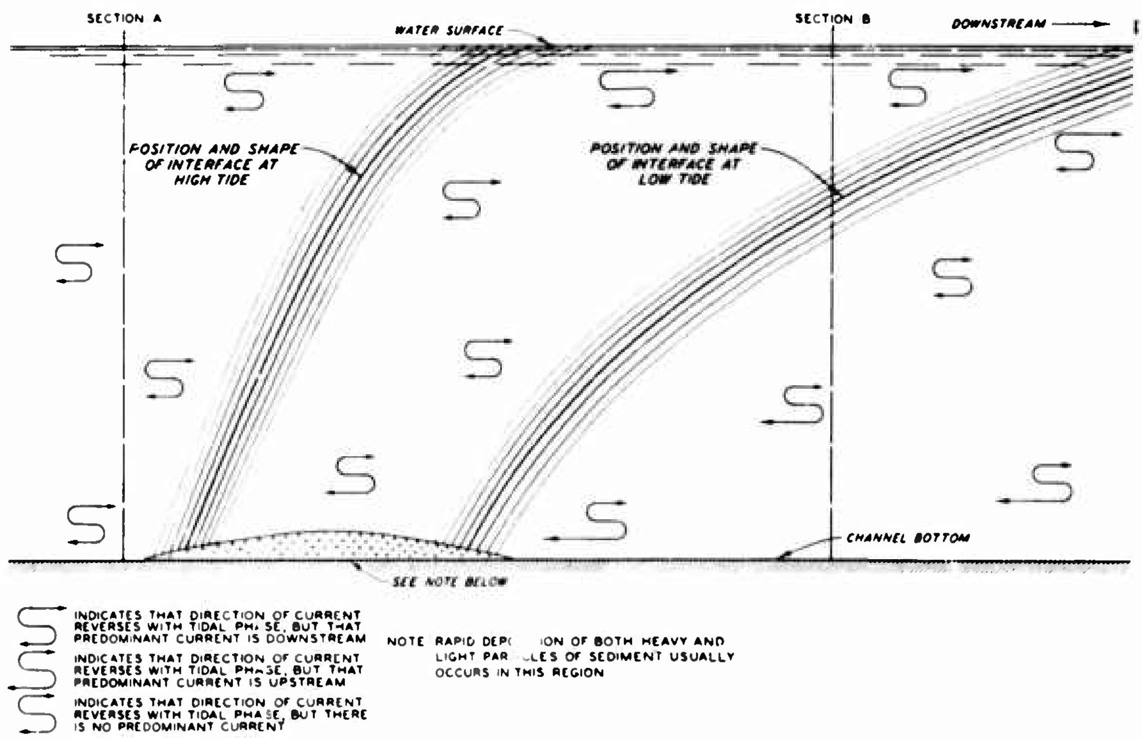


Fig. V-2. Conditions typical of partly mixed estuary intrusion limit and the lighter particles just downstream. As the location of the tip of the wedge varies in location with variations of upland discharge, shoaling may take place over a considerable reach of channel.

V-7. Fig. V-2 shows schematically the distribution of current velocities and directions in a partly mixed estuary, which is probably the most common type found in nature. Typical examples of the partly mixed estuary are the Hudson River, Charleston and Savannah Harbors, and the St. Johns River. The interface in the partly mixed estuary is not so well defined as that in the highly stratified type, but it can be detected by either salinity or current velocity measurements. The interface in the partly mixed estuary moves upstream and downstream as the tide floods and ebbs, usually over a distance of several miles. Although the currents at all depths usually reverse with tidal phase, the downstream currents in the fresher water strata are much stronger than the upstream currents, while the upstream currents in the more saline water strata are stronger than the downstream currents. As in the case of the highly stratified estuary, the maximum currents in the fresher water are usually near the surface and near the estuary entrance, while those in the more saline strata are usually near the bottom and near the upstream limit of saltwater intrusion. Measurements show that the salinity at the bottom is appreciably greater than at the surface, especially at low tide, and that the greatest change in salinity from surface to bottom occurs in about 20 percent of the total depth. Because of the greater density of the water in the lower strata, the reversal in flow from ebb to flood occurs earlier at the bottom than at the surface with the result that the duration of the bottom flood current exceeds that of the bottom ebb current. At the surface the duration of the ebb current exceeds that of the flood because the fresh water is exhausted from the estuary principally in the surface strata.

V-8. The region of heaviest shoaling in the partly mixed estuary frequently lies between the high-tide and low-tide positions of the upstream limit of saline intrusion as indicated in fig. V-2, but some cases may extend to the mouth of the estuary, depending on certain conditions which will be explained later. As in the case of the highly stratified estuary, the heavier particles of sediment come to rest when they reach the intrusion limit. The lighter particles may be carried well down into the estuary before they enter the predominantly upstream flow in the lower strata, and in some cases they are then transported back upstream to the vicinity of the intrusion limit. There is little evidence of sorting of the sediment particles in the partly mixed estuary, since the intrusion limit moves back and forth over an appreciable distance with tidal phase which permits intermingling of the sediment particles.

V-9. Shoaling in Charleston Harbor, which is an example of the partly mixed estuary, is heaviest in isolated reaches far downstream from the range of movement of the interface. In cases such as this, the controlling features appear to be related to local interruptions in the predominance of upstream current on the bottom in combination with such factors as excessive cross-sectional area, eddies and crosscurrents, and other physical features.

V-10. The distributions of current velocities and directions in the well-mixed estuary, of which the Delaware and Raritan Estuaries provide typical examples in nature, have similar characteristics to those of the partly mixed estuary. The mixing of salt water and fresh water is of a high degree in this type of estuary, with the result that a definite interface rarely exists. However, surface salinities are somewhat less than bottom salinities at any given location, and careful analysis of current velocity measurements will usually show characteristics similar to those of the partly mixed estuary though much less prominent. Ebb velocities predominate over flood velocities in the upper strata, but flood velocities may predominate slightly over ebb velocities at the bottom. A detailed analysis of the velocity data, by methods which will be discussed subsequently, is often necessary to ascertain whether this condition exists.

V-11. The rate of change in salinity from surface to bottom is nearly constant; a rapid change at some critical depth is not evident as in the case of the partly mixed estuary. However, bottom salinities are usually some 15 to 25 percent greater than surface salinities, and this difference is sufficient to affect the vertical distribution of current velocities to some extent. The characteristics of the velocity distribution in the well-mixed estuary are similar to those of the partly mixed type, the difference between the two being almost completely in the degree of density effect.

V-12. Since the effects of density differences on the vertical distribution of currents are small in the well-mixed estuary, the distribution of shoaling in this type does not appear to be directly related to the limits of salinity intrusion except in isolated instances. The locations of shoals in the well-mixed estuary appear to be influenced less by the effects of salinity intrusions on currents than by such factors as excessive cross-sectional area, nonuniform flow caused by islands and divided channels, and other physical features that affect transport velocities.

V-13. It is probable that the two principal factors that dictate whether an estuary is highly stratified, partly mixed, or well mixed are (a) upland discharge, and (b) turbulent mixing of fresh water and salt water in the estuary by tidal action. Normally, the highly stratified estuary is found where the upland discharge is large with respect to the tidal forces, the partly mixed estuary where the upland discharge and the tidal forces are approximately equal, and the well-mixed estuary where the upland discharge is small with respect to the tidal forces.

V-14. The mean tidal prism of an estuary (defined as the net volume of water which would flow into the estuary from the sea during an average flood-tide period with no upland inflow) provides a fair index to its tidal forces. Simmons has found from experience that the ratio of mean upland discharge into an estuary (defined as the volume of upland water entering an estuary for conditions of mean upland discharge and over the interval of a mean tidal cycle of about 12.42 hr) to its mean tidal prism is a fairly dependable index to its mixing type. Where the ratio is of the order of 1.0 or greater (average for Southwest Pass of the Lower Mississippi River is about 1.25), the highly stratified type is normally found; where the ratio is of the order of 0.25 (average for Savannah Harbor is about 0.20), the partly mixed type is normally found; and where the ratio is appreciably less than 0.1 (average for the Delaware Estuary is about 0.01), the well-mixed type is normally found. This ratio should be used only in a very general way, since there is evidence that one reach of a given estuary may be partly mixed while another reach may be well mixed for identical conditions of tide and upland discharge, but it does provide a quick and fairly dependable means for predicting estuary mixing type in the absence of detailed measurements of current velocities and salinities. The tabulation below shows for San Francisco Bay the classifications of different parts of the Bay system, and changes in classification for different freshwater inflows into the Bay system (stations are shown in fig. V-3).

Portion of System	Freshwater Inflow 5000 cfs	Freshwater Inflow 45,000 cfs (mean)
North & south of sta 8	Well mixed	Well mixed
Southeast of sta A	Well mixed	Well mixed
South of sta S-4	Well mixed	Well mixed
North of sta G	Well mixed	Well mixed
East of sta I	Well mixed	Partly mixed
East of sta K	Well mixed	Partly mixed
	100,000 cfs	**200,000 cfs**
North & south of sta 8	Well mixed	Partly mixed
Southeast of sta A	Well mixed	Partly mixed
South of sta S-4	Partly mixed	Partly mixed
North of sta G	Partly mixed	Partly mixed
East of sta I	Partly mixed	Highly stratified
East of sta K	Highly stratified	Highly stratified

V-15. If we can assume that upland discharge into an estuary plays an important role in dictating whether it will be highly stratified, partly mixed, or well mixed, then the importance of prior knowledge indicating the effects of

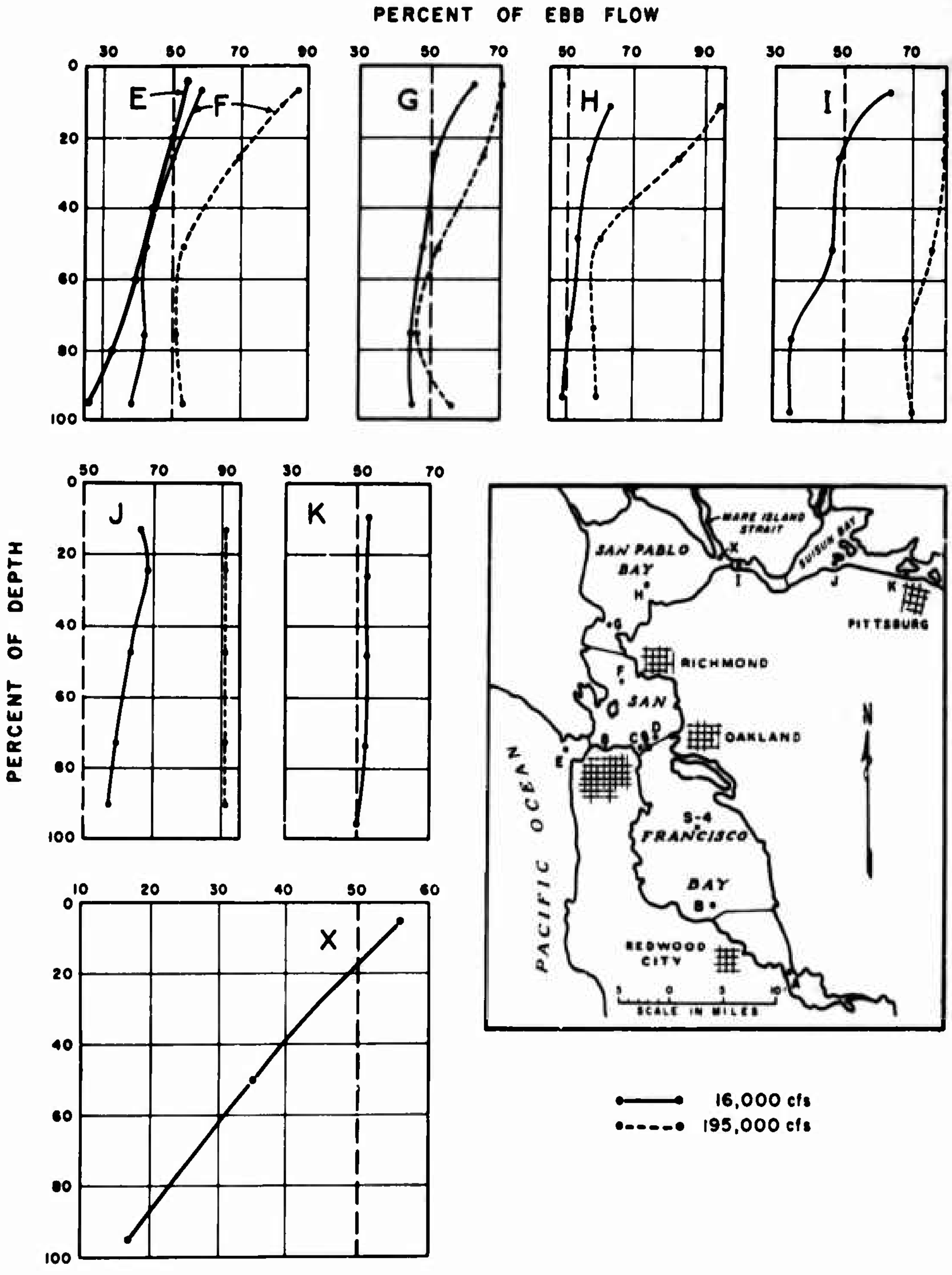
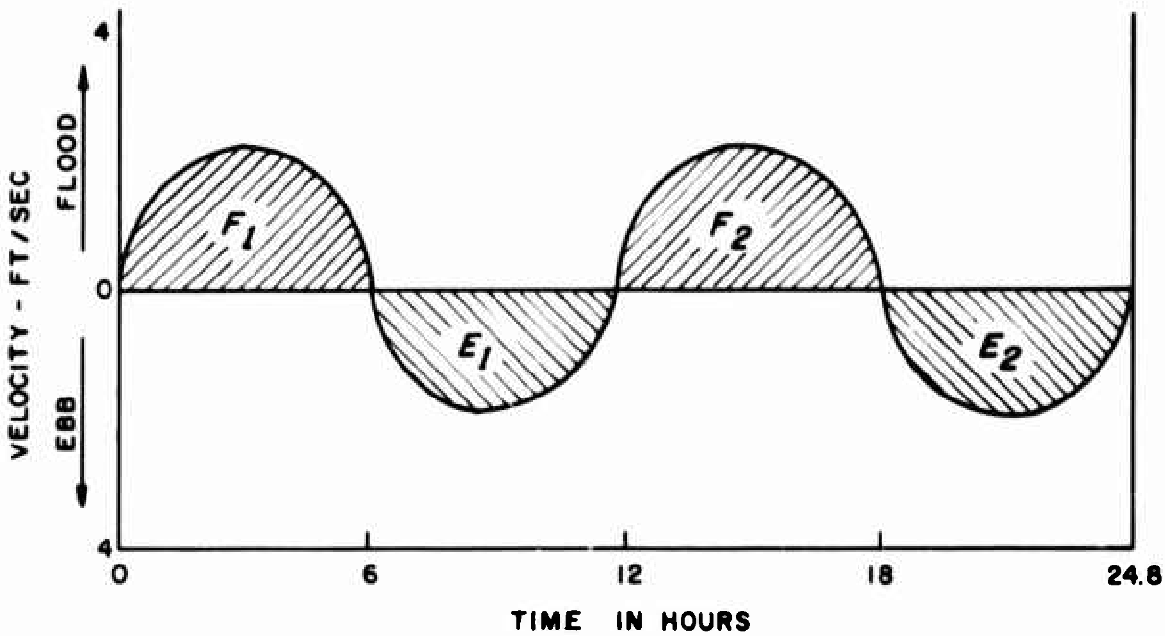


Fig. V-3. Predominance curves, San Francisco Bay changing the upland discharge into an estuary is obvious. For example, an existing partly mixed estuary may be changed to well mixed by reducing the upland discharge, or it may be changed to highly stratified by increasing the upland discharge. Similarly, a change in the tidal establishment may, in some cases, be sufficient to cause modification of the degree of mixing. Since information available indicates that the pattern of shoaling in estuaries varies with type mixing, it follows that estuarine sedimentation is an important effect of upland discharge.

V-16. In considering the effects of changes in upland discharge on the hydraulics of an estuary, and in turn on its shoaling characteristics, the investigator is confronted with a large number of factors that may be used for analysis; for example, a change in upland discharge may be considered in terms of its effect on maximum flood or ebb velocities, mean flood or ebb velocities, duration of flood or ebb currents, etc. But it is very difficult to interpret the effects of changes in upland discharge on any one hydraulic factor in terms of change in shoaling characteristics. The sediments that comprise shoaling in estuaries often are clays and other fine-grain materials which, once placed in suspension, are transported readily by very weak currents. Experience with shoaling in estuaries, especially in navigation slips and dead-end channels, has led to the opinion that such sediments will be transported anywhere that water goes. For this reason, establishment of a single criterion to define the hydraulic characteristics of an estuary, and one that can be used in turn to define the principal features of its resulting shoaling pattern, is very important.

V-17. Within the past few years, Simmons has developed a method for analyzing current velocity measurements that offers promise as a means for defining the hydraulic characteristics of an estuary in simple terms, describing the broad features of the accompanying shoaling pattern, and predicting with fair accuracy the effects of proposed changes in upland discharge or tidal prism on hydraulic and shoaling characteristics. In essence, this method reduces data on the velocities, directions, and durations of the currents into a single expression that defines the direction and degree of predominance at any given location and depth. Let it be assumed that current velocity measurements are made at 10 percent increments of depth at a given station over a complete tidal cycle. A conventional plot of velocity versus time for each point in the vertical is then made (fig. V-4). The areas subtended by both the flood and ebb curves of the plot are planimetered, and the total area thus determined provides an index to the total flow at the point of measurement. The area enclosed by the ebb curve is then divided by the combined areas under the flood and ebb curves, and the resultant determines what percentage of the total flow at the point of measurement is downstream. From the value thus determined



$$\frac{E_1 + E_2}{F_1 + F_2 + E_1 + E_2} = \text{\% OF FLOW DOWNSTREAM}$$

Fig. V-4. Computation of flow predominance from each point in the vertical, a second plot is constructed to define the distribution of the total flow between flood and ebb from surface to bottom for the vertical in question; this plot defines the direction and degree of predominance of flow from surface to bottom. It is not in any sense a graph showing the distribution of discharge from surface to bottom at the vertical involved, nor over the entire cross section.

V-18. Fig. V-3 shows San Francisco Bay and flow predominance curves obtained by the method described in the preceding paragraph. Except for station X at the entrance to Mare Island Strait, current velocity measurements were made in the deepwater channels of the prototype for conditions of approximate mean tide at stations extending from outside of Golden Gate (station E) to a point nearly 50 miles inside (station K) for a low freshwater inflow into the Bay system of 16,000 cfs (dry season flow) and a high freshwater inflow of 195,000 cfs (wet season flow). Measurements at station X were made in the hydraulic model of the Bay system for the low freshwater inflow.

V-19. With minor exceptions, the high freshwater inflow caused ebb flow to predominate throughout the depth from Golden Gate to the eastern end of Suisun Bay. The low freshwater inflow caused flood flows to predominate in the bottom strata from Golden Gate to about the middle of Carquinez Strait (east of station I), and at most stations this predominance prevailed in the lower 75 to 80 percent of the depth. In Mare Island Strait, at its mouth, and in areas of San Pablo Bay and Carquinez Strait adjacent to the mouth, it is found that the flood current predominated during a wide range of upland discharges, and that this characteristic was exhibited to a considerable degree throughout a large portion of the depth. In fact, at two stations flood predominated throughout the entire depth.

V-20. The current characteristics above explain why the Mare Island Strait navigation project experiences the largest amount of shoaling of all navigation projects within the Bay system. During the higher freshwater flows that occur more frequently, flood predominates along the bottom in San Pablo Bay and seaward movement of sediment in the bottom strata is interrupted, so this bay is a sediment trap most of the time. Fortunately, at times of the rare great river floods, when the largest sediment loads are delivered to the bay, ebb predominates throughout the depth throughout the entire upper Bay system, and consequently the net transport is seaward; otherwise, shoaling within and without the navigation channels would be much larger than experienced. Nevertheless, part of the sediment load brought down during great river floods settles out in areas favorable to deposition, such as the shallows and areas of eddies and stagnation, and during slack periods of current, and thus remains as a potential source of material, subject to disturbance by waves and movement by tidal currents, to shoal the channels. San Pablo Bay is the largest local source of material that causes shoaling in Mare Island Strait, evidenced by the fact that shoaling in the strait is most rapid during the dry season when the river flows and sediment loads are small, and as indicated by reconnaissance-type field observations and radioactive tracer tests.

V-21. Fig. V-5 presents flow predominance curves for various locations in Savannah Harbor (fig. V-6), and fig. V-7 is an analysis of the data in fig. V-5. It shows the percentage of total flow downstream at the surface and bottom for each station plotted against channel stations as abscissae. The graphs indicate those regions of the harbor in which the currents at the depths under consideration are predominantly upstream or downstream and in what degree.

V-22. Sufficient prototype data to show how the flow distribution of a given estuary is affected by upland discharge over a wide range of conditions are rarely available. Fortunately, a series of current velocity measurements was made in Savannah Harbor a few years ago, and data obtained at one point were sufficiently complete to permit a detailed analysis of flow distribution over a wide range of upland discharge. The results of this analysis, which show the predominance of flow at surface, middepth, and bottom for upland discharges ranging from a low of about 3000 cfs to a high of 66,000 cfs, are shown in fig. V-8. The flow distribution curves in fig. V-8 indicate that for zero upland discharge the flow is almost exactly 50 percent downstream and 50 percent upstream at all depths. Without upland discharge, there is no predominance of flow in either direction at any depth, because there is no possibility of stratification. As the upland discharge increases, flow in the surface strata is more and more predominantly downstream until at an upland discharge of about 40,000 cfs the direction of flow in the surface strata is downstream throughout the tidal cycle. At middepth, the percentage of total flow downstream increases from 50 to about 55 as the upland discharge increases from zero to about 10,000 cfs. Flow in these strata remains about 55 percent downstream until that in the surface strata is 100 percent downstream, following which there is a sharp increase in the predominance of downstream flow at middepth. For an upland discharge of 66,000 cfs, flow in the middepth strata is about 75 percent downstream. At the bottom, the percentage of total flow downstream decreases sharply from 50 percent at zero upland discharge to a minimum of about 25 percent for an upland discharge of about 20,000 cfs. Further increases in upland discharge above 20,000 cfs gradually increase the percentage of downstream flow in the bottom strata; however, for a discharge of 66,000 cfs the total flow downstream in the

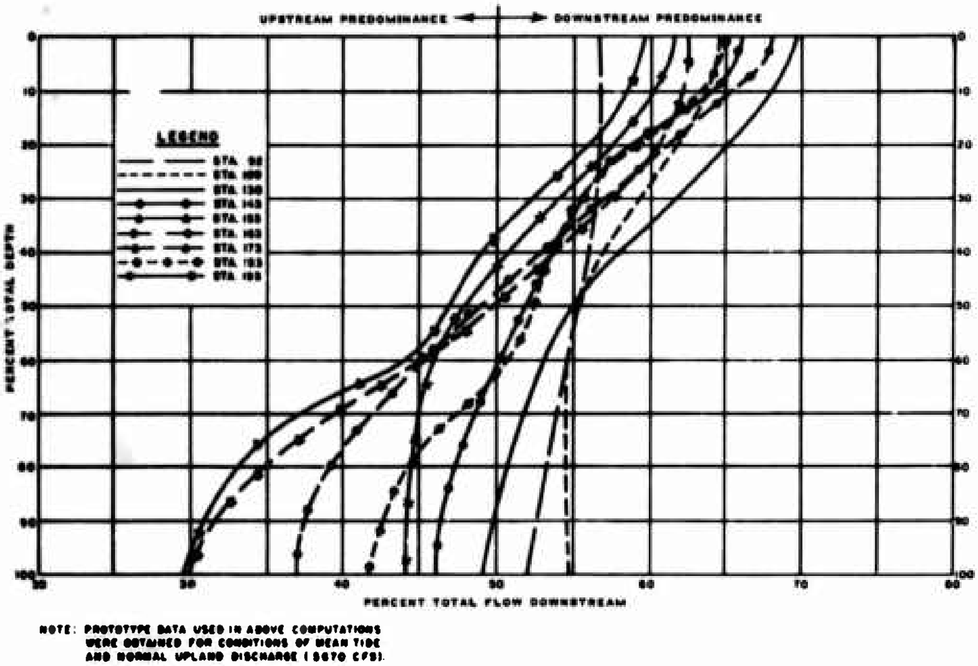


Fig. V-5. Flow predominance curves, Savannah Harbor

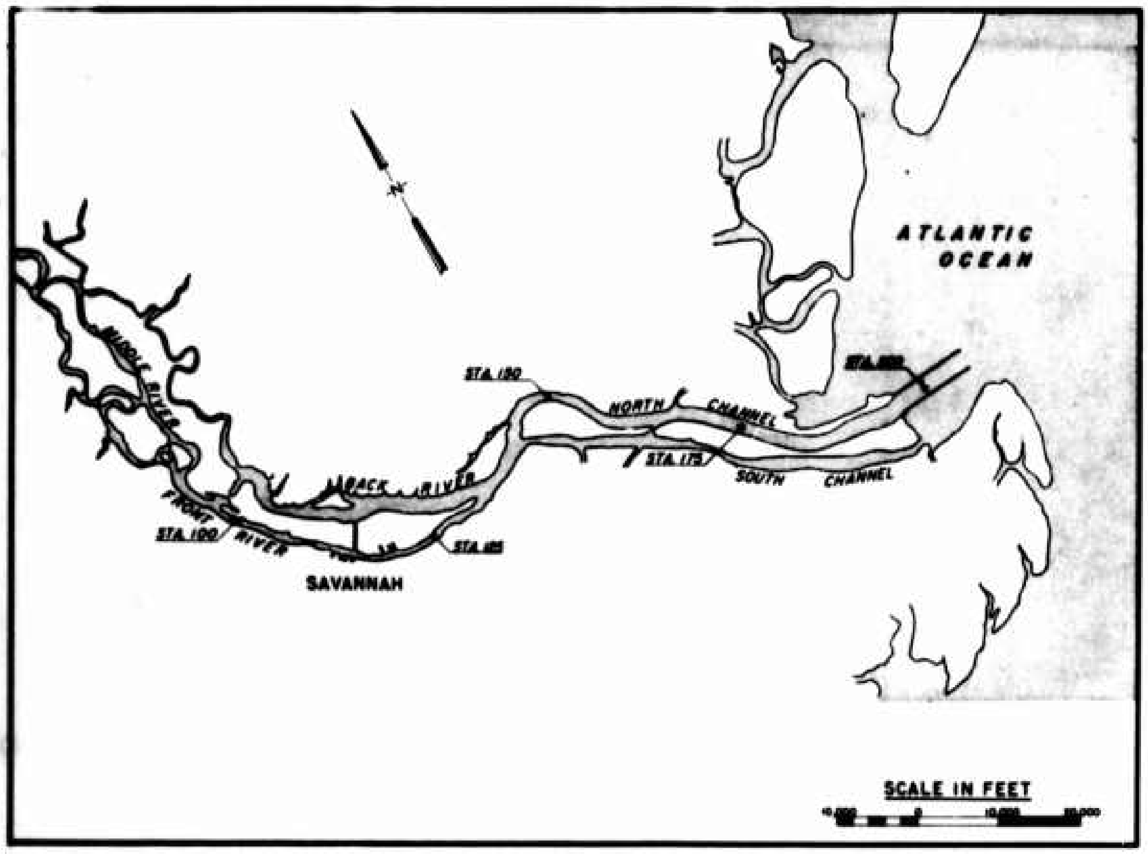
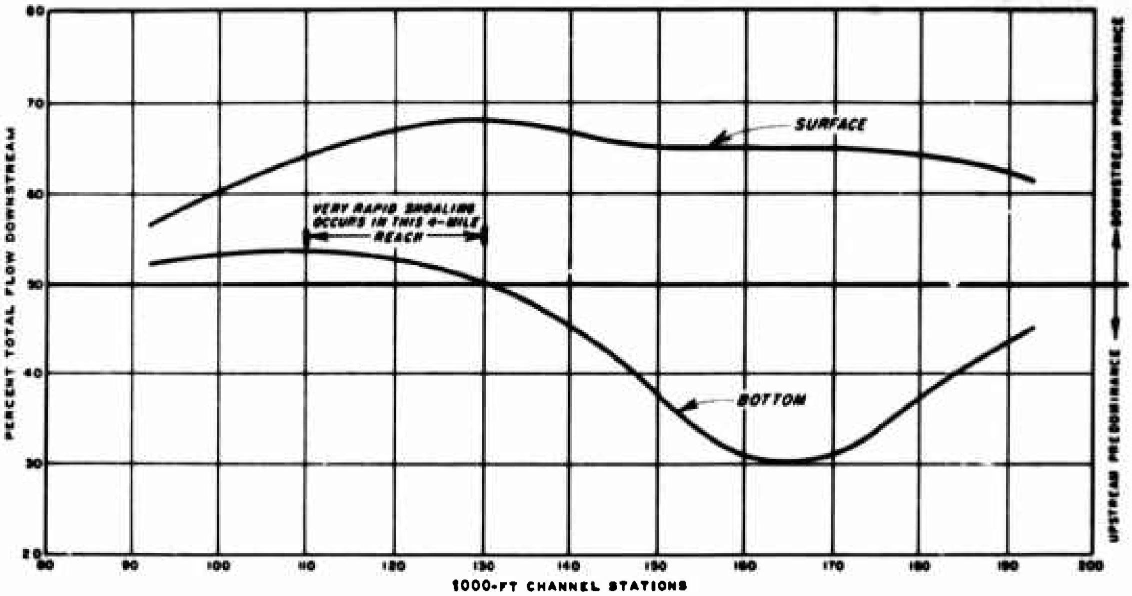


Fig. V-6. Location map, Savannah Harbor



NOTE. PROTOTYPE DATA USED IN ABOVE COMPUTATIONS WERE OBTAINED FOR CONDITIONS OF MEAN TIDE AND NORMAL LOW UPLAND DISCHARGE (5670 CFS).

Fig. V-7. Predominance of flow in surface and bottom strata, Savannah Harbor

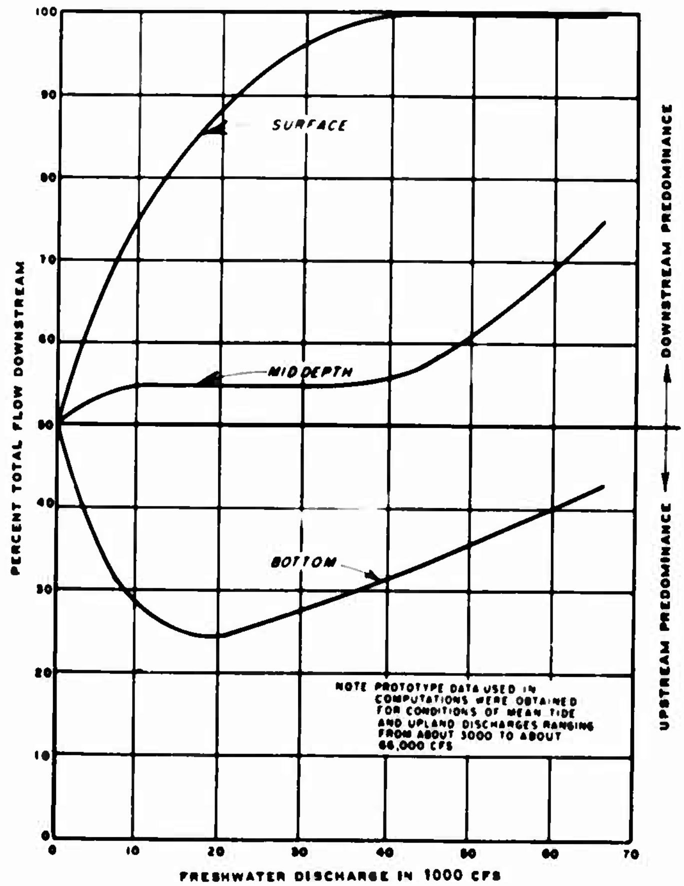


Fig. V-8. Effects of upland discharge on predominance of flow, station 190, Savannah Harbor bottom strata is only about 43 percent as compared to 50 percent for zero discharge.

V-23. Study of these data has led to the following conclusions: (a) for the conditions of tide and upland discharges considered in this analysis, sediment entering the upstream end of Savannah Harbor is moved progressively downstream at all depths to the vicinity of station 130; (b) sediments transported by the bottom strata will be deposited in the vicinity of station 130, since the predominance of upstream flow along the bottom throughout the remainder of the harbor will prevent further net movement of such sediments toward the sea; (c) sediments carried by the upper strata will be transported toward the sea, but a major portion of these sediments will sink into the bottom strata during intervals of slack current and will be carried upstream to the vicinity of station 130 by the predominant upstream flow in the bottom strata. These deductions are confirmed by maintenance dredging records for Savannah Harbor showing that by far the largest shoal in the harbor is located at and upstream from station 130. It has been determined that the rate of shoaling in this reach of channel is greatest for freshwater discharges of the same order of magnitude as that occurring at the time of the field velocity measurements used in the above analysis. Greater or smaller freshwater discharges will shift the location of the point of zero predominance of bottom flow (50 percent total flow downstream) in the harbor and thus shift the area of most rapid shoaling. A few years ago, the freshwater discharge into Savannah Harbor was held at about 20,000 cfs for a period of several months by operation of the outlet works at Clark Hill Dam, and it was noted that no shoaling occurred in the vicinity of station 130 during the entire period, especially in the upstream portion of the shoal area.

V-24. Fig. V-9 presents for Charleston Harbor, S. C., the results of computations of the percentage of total flow downstream at the surface and at the bottom for the various velocity stations plotted against distance in miles along the channel for conditions of mean tide and average freshwater discharge (15,000 cfs). As in the case of Savannah Harbor, the resultant curves indicate those regions of the harbor in which the currents at the depths under consideration are predominantly upstream or downstream and in what degree. The surface flow throughout the entire harbor is predominantly downstream, the percentage of total flow in this direction ranging from 65 to 80 percent. The bottom flow throughout the lower harbor from the jetties to a point about 11 miles upstream (except for a 3/4-mile reach below the mouth of Wando River) is predominantly upstream, the predominance increasing progressively with distance upstream. At mile 9.5, which is the location of

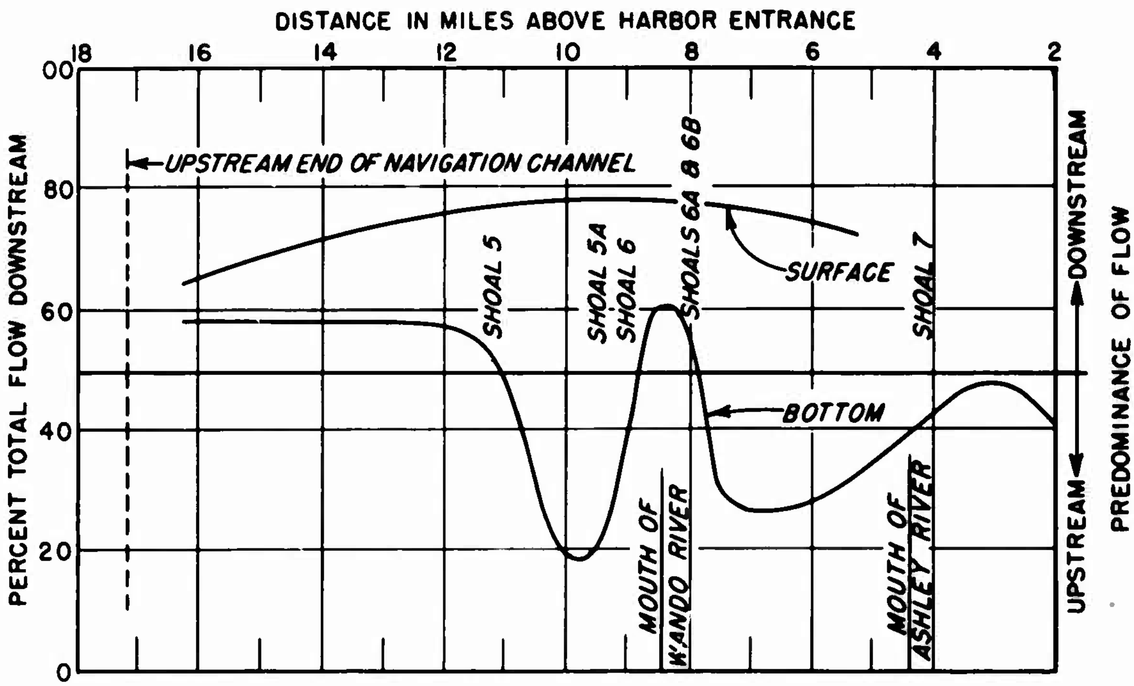


Fig. V-9. Predominance of flow in surface and bottom strata, Cooper River the second largest shoal in the harbor, only 15 percent of the total flow at the bottom is downstream. Between mile 17 and mile 11 the bottom flow is predominantly downstream; this changes from downstream to upstream between miles 11 and 9 and again to downstream predominance in the 3/4-mile reach below Wando River. The largest shoals in Charleston Harbor, in terms of volume, are found in the lower harbor, and maintenance dredging records reveal that the bulk of the dredging is performed in the 11 miles of channel where bottom flood currents predominate (fig. V-10). The smallest shoals are located in the upper 6 miles, where the bottom ebb currents predominate. The fact that some shoaling occurs in the latter reach is attributed to the upstream and downstream shifting of the freshwater-saltwater interface under variable conditions of tide and freshwater discharge, and to other factors such as excessive cross-sectional area. The coarsest grained sand is found in the shoals in this region of the harbor.

V-25. As stated previously, the geometry of tidal waterways may modify currents directly and thereby cause shoaling, or it may have an effect on the pattern of salinity intrusions and this in turn will affect the currents and thus cause shoaling. A tributary stream contributing a significant quantity of fresh water may modify the characteristics of the salinity intrusion in the main waterway from well mixed to stratified. On the other hand, a large arm of the estuary not having a significant freshwater inflow may abruptly modify the tidal discharges of the main waterway and possibly change the characteristics of the salinity intrusion from highly stratified or partly mixed above the mouth of the arm to one having a greater degree of mixing below the mouth of the arm. A marked constriction will accelerate currents and produce additional turbulence, which enhances the mixing of the salt water and fresh water. It is possible for this condition to eradicate all density differences at and adjacent to the constriction, and thus change an upstream predominance of the currents near the bottom to a downstream predominance. The significance of these and other modifications in the characteristics of the salinity intrusions lies in the obvious fact that it cannot be assumed that a given waterway has the same kind of salinity intrusion throughout the length of waterway ordinarily experiencing salinity intrusions.

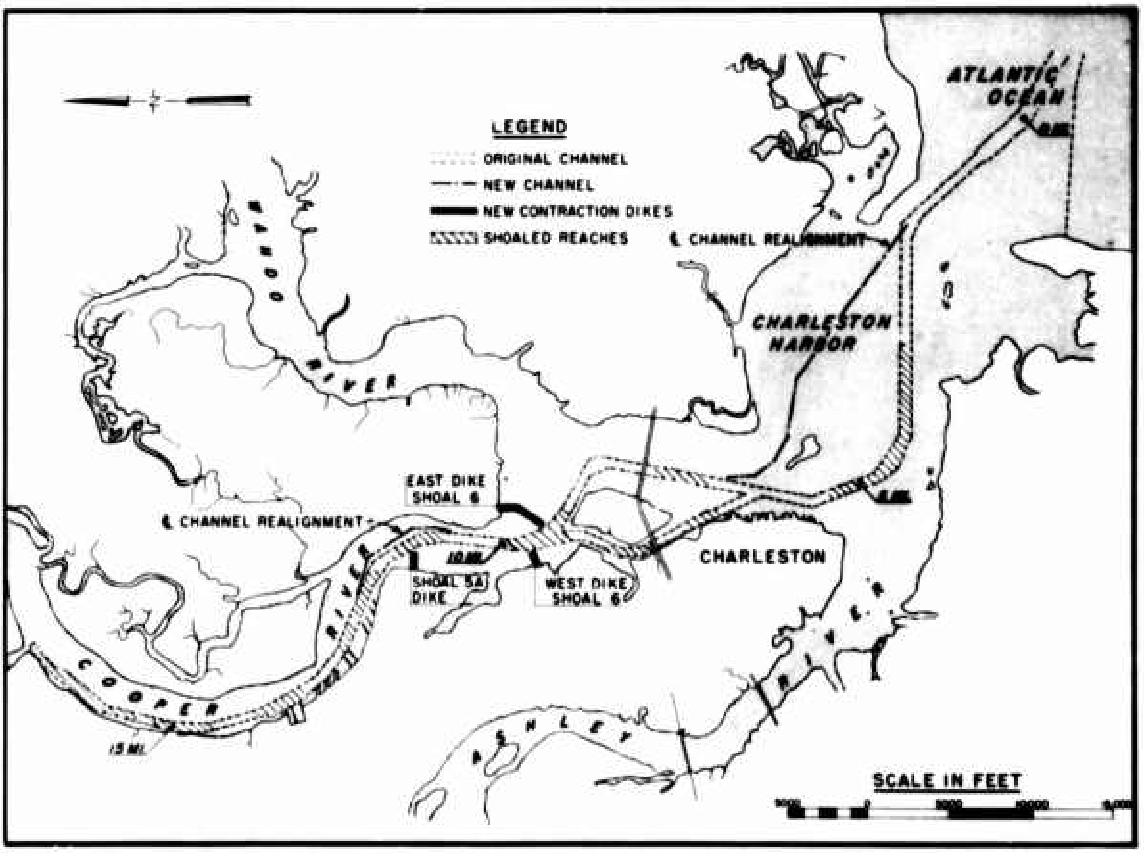


Fig. V-10. Location map, Charleston Harbor

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CHAPTER VI

EFFECTS OF LITTORAL PROCESSES ON TIDEWATER NAVIGATION CHANNELS

by

J. M. Caldwell and J. B. Lockett

Introduction

VI-1. This chapter discusses the movement of beach materials along the shore face and into the mouths of estuaries, inlets, and bays as the result of littoral processes. It emphasizes the importance of determining the source and magnitude of these alongshore movements and their effect on the securing and maintenance of navigation channels. The functions and effectiveness of corrective works, including sand bypassing plants, are discussed as bases for consideration and selection of the most feasible and practical expedient to be employed for achieving the goal desired.

Definitions

VI-2. In this chapter, the following definitions apply:

a. An estuary is considered to be that portion of a stream which is subject to periodic variations in water levels as influenced by the rise and fall of the ocean tide, and which discharges an appreciable quantity of fresh water from an inland drainage area.

b. An inlet is considered to be a short, constricted channel between the ocean and an interior lagoon. The constriction is usually sufficient to prevent the free passage of tidal action, and the tidal range in the lagoon or bay is significantly less than that in the ocean outside the inlet.

c. A lagoon is defined as a body of water connected to the ocean by an inlet. The tidal range in a lagoon is less than in the adjacent ocean and tidal currents in the bay are much less than in the inlet passage to the ocean.

d. A wave is a moving ridge or swell on the surface of the water normally generated by wind action.

e. Density currents are currents produced by differences in specific gravity such as occur in estuaries where the lighter freshwater discharge meets the heavier salt water.

f. Littoral processes relate to the phenomenon of movement of ocean water masses and beach materials along a shore resulting from the incidence of waves approaching the beach at an angle, which, in combination with the hydrography of the shore, produces alongshore littoral currents. These particular currents act as transportation agents for the movement of beach materials along a shore, referred to as littoral drift.

VI-3. A proper investigation of a shoaling problem at an entrance to an estuary, or in an inlet, includes consideration of the possible sources of the material in the shoal. It is unlikely that an engineering structure designed to reduce shoaling will be effective without an analysis of the forces and conditions under which it will operate. The source of the material in the shoal and the mechanics whereby it is transported from that source to the shoal, and there accumulates, is a primary consideration. In many cases it may quickly become obvious that the sources of the shoal materials are the beaches flanking the inlet.

VI-4. Shoaling particles retain characteristics of materials from which they originate and much can be learned by careful analysis of the different physical, chemical, and other qualities they possess. Materials having their source in upland areas are ordinarily borne by streams either in suspension in the fresh water or rolled along the bottom of the stream. Some upland sediments, due to their chemical composition, undergo a process known as flocculation and may settle out upon reaching brackish or salt water to form part of the shoaling mass.

VI-5. Materials originating in offshore areas also possess characteristics of their source. Usually, evidence of a littoral movement of materials may be readily ascertained through a study of hydrographic maps and aerial photographs of the area which will normally indicate a greater degree of natural shoaling or buildup of materials along the one side of an entrance. Shoaling developing as the result of littoral processes will generally possess characteristics of material eroded by wave action in updrift areas. In the event currents within the entrance are of sufficient magnitude, these materials from immediate offshore bottom areas are brought into the entrance. Wave erosion of adjacent headlands is capable of causing or aggravating shoaling conditions within an entrance channel.

VI-6. Through study of the above possibilities, together with laboratory analysis of the structure and composition of the shoal material, it may be possible to determine within an acceptable degree of accuracy the source of shoaling materials. Shape and size of shoal particles often denote the conditions under which the material has been formed, as for example, rounded particles are generally indicative of sea action over an extended period of time, while pointed or sharp particles usually result from comparatively recent river erosion of land formations. Chemical properties of shoal materials can sometimes be positively identified with certain land formations or industrial waste operations along tributary streams. The weight of shoal materials per unit of volume is a significant indication of source, particularly along certain streams.

Wave origin and generation

VI-7. The normal ocean wave results from the action of wind upon the water. Although the mechanics by which the wind transfers energy to the water is not clearly understood, it is known that the wind velocity, its duration, and the "fetch," or the distance over which a wind development is actually in contact with the water surface, are prime factors which determine the size of the generated waves. In such waves, classed as progressive oscillatory waves, the water particles in deep water travel in circular paths and remain more or less in the same locality.[3*] The wave form, once developed, travels progressively forward and is capable of propagating itself great distances. For instance, some of the waves reaching the shores of southern California have their origin several thousand miles distant in the south Pacific Ocean below the equator.

VI-8. Individual waves have characteristics of height (the vertical distance from crest to trough), length (the distance from crest to crest), and period (the time required for successive crests to pass a given point). Fig. VI-1 shows certain wave characteristics and it should be noted that the wave length, wave period, and water depth are interrelated.

VI-9. A wave train is a series of wave crests traveling in sequence. As the individual waves constituting such a movement are not of uniform height or length, there is a dissimilar collection of waves present in the train which may be considered to form a "spectrum" of various types, including high and low waves as well as long and short waves in many combinations. Generally, however, some particular set of waves of certain height and period appear to predominate throughout the train and the average of these dominant waves is considered to represent the significant wave of the wave train. Although the procedure used in determining the significant height and significant period from a graphical record of the individual waves of a wave train is fairly complicated, it might be described as averaging about the higher one-third of the dominant waves of the wave train. The average wave period associated with these significant waves is referred to as the significant period. While there is no universal acceptance of the significant wave concept to describe the characteristics of a wave train, this concept is presently widely used as an index of wave train action.

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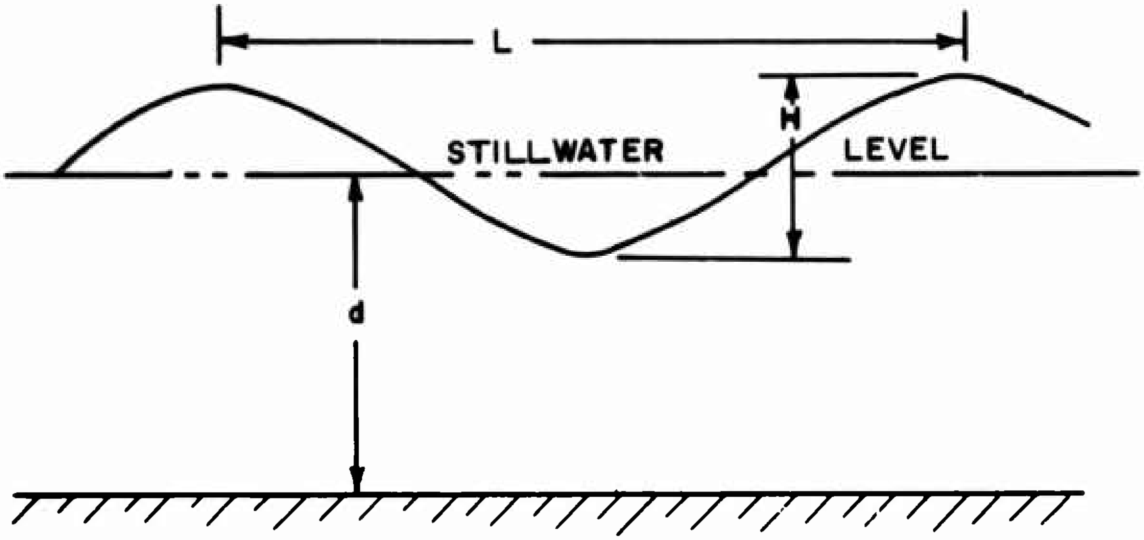
* Raised numerals refer to similarly numbered items in Selected Bibliography at the end of this chapter.



L = WAVE LENGTH

H = WAVE HEIGHT

d = MEAN WATER DEPTH

C = WAVE VELOCITY

T = WAVE PERIOD

GENERAL EQUATION : $C = \sqrt{\frac{gL}{2\pi} \operatorname{Tanh} \frac{2\pi d}{L}}$

APPROXIMATE EQUATION FOR $d/L > 1/2$: $C = \sqrt{\frac{gL}{2\pi}}$

APPROXIMATE EQUATION FOR $d/L < 1/25$: $C = \sqrt{gd}$

$L = CT$

Fig. VI-1. Certain wave characteristics

VI-10. The forecasting of wave action at any given locality is generally accomplished through the use of wind data obtained from weather maps, particularly those presenting an analysis of observations taken in various places over a wide region at or near the same time. A person trained in the interpretation of such weather maps can readily recognize which portion of the wind field will generate waves that will move toward the locality under consideration. It will also be possible from these data to predict the velocity, duration, and fetch of the wind and thus the significant period and height of the wave train moving toward the locality. A few of the relations used in wave forecasting, once these characteristics have been determined, are shown in fig. VI-2.[6]

Wave decay, transition, refraction, diffraction, and steepness

VI-11. Waves generated by local wind conditions generally reach the shores of lakes, reservoirs, and inland waters with essentially the same magnitude of energy imparted in the wave-generating area. On the other hand, ocean wave trains reaching a shore from a wave-generation area hundreds or thousands of miles away lose a significant portion of their original energy. The distance between the end of the fetch and the shore on which the waves impinge is known as the decay distance. Movement of the wave train through the decay distance results in the dropping behind and fading out of the shorter waves and the assumption of increasing prominence by the longer waves which produces a change in significant height and period of the wave train. Fig. VI-3[6] illustrates the procedures used in determining the significant wave height and period at the end of the decay distance, provided the decay is not complicated by intermediate winds or currents.

VI-12. As long as the water depth is greater than one-half the wave length, waves are not appreciably influenced by water depth. As they approach shallow water, however, waves tend to diminish in velocity, shorten in length, and alter the circular movement of the wave particles to an elliptical path.[3] Also, the wave increases in height as its length is shortened until it reaches a depth so shallow that it is unable to transmit itself forward as a progressive oscillatory wave. At this point, the wave breaks and continues toward the beach as a breaker, the breaker being essentially a translatory wave. At times, a breaker may reform into an oscillatory wave and then break again before reaching the shore. Fig. VI-4 shows the relations for determining the heights of breakers and the breaking depth when the deepwater wave characteristics are known and wave refraction is not involved.

VI-13. When deepwater waves approach a shore at an angle to the beach

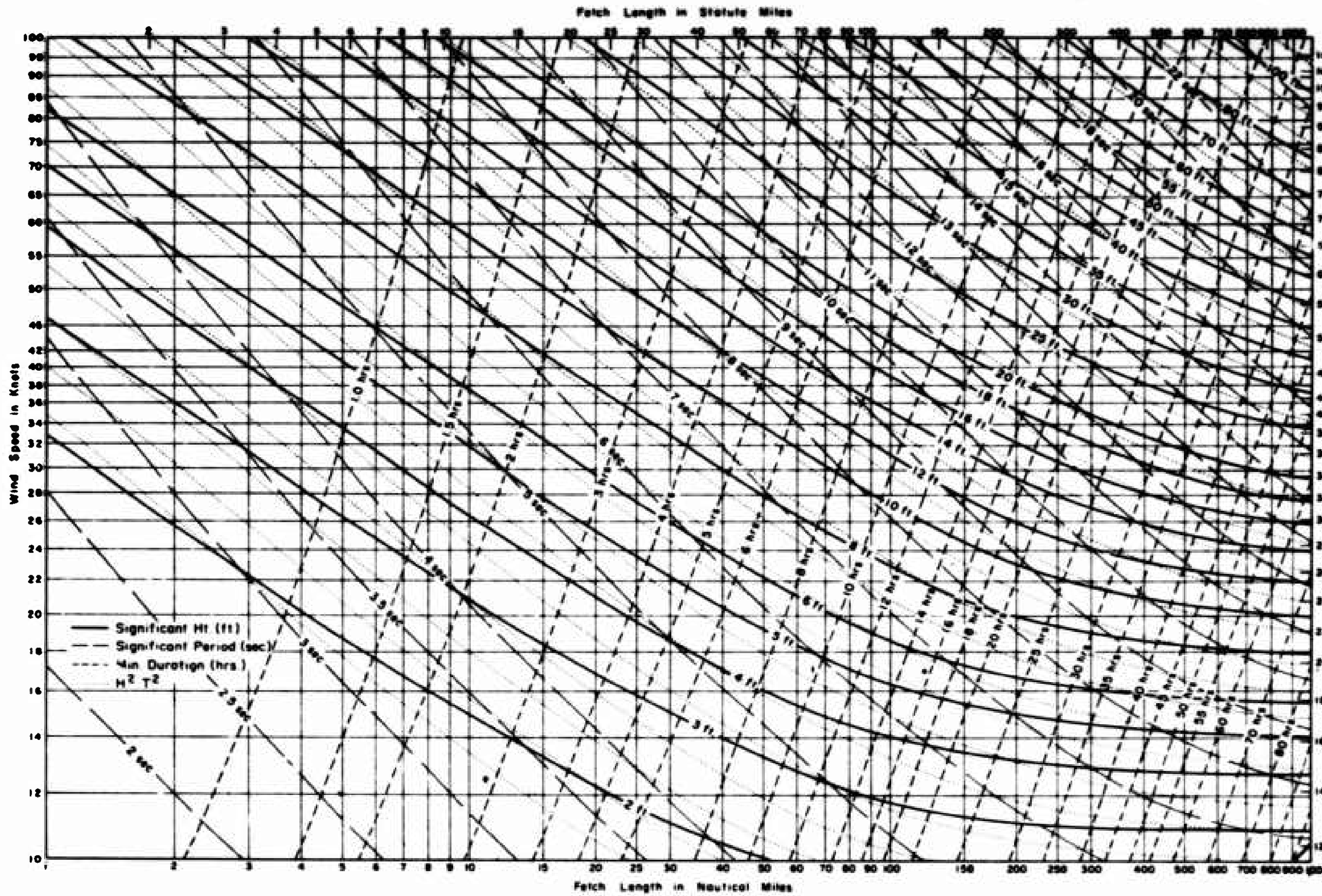


Fig. VI-2. Deepwater wave forecasting as a function of wind speed, fetch length, and wind duration (for fetches 1 to 1000 miles)

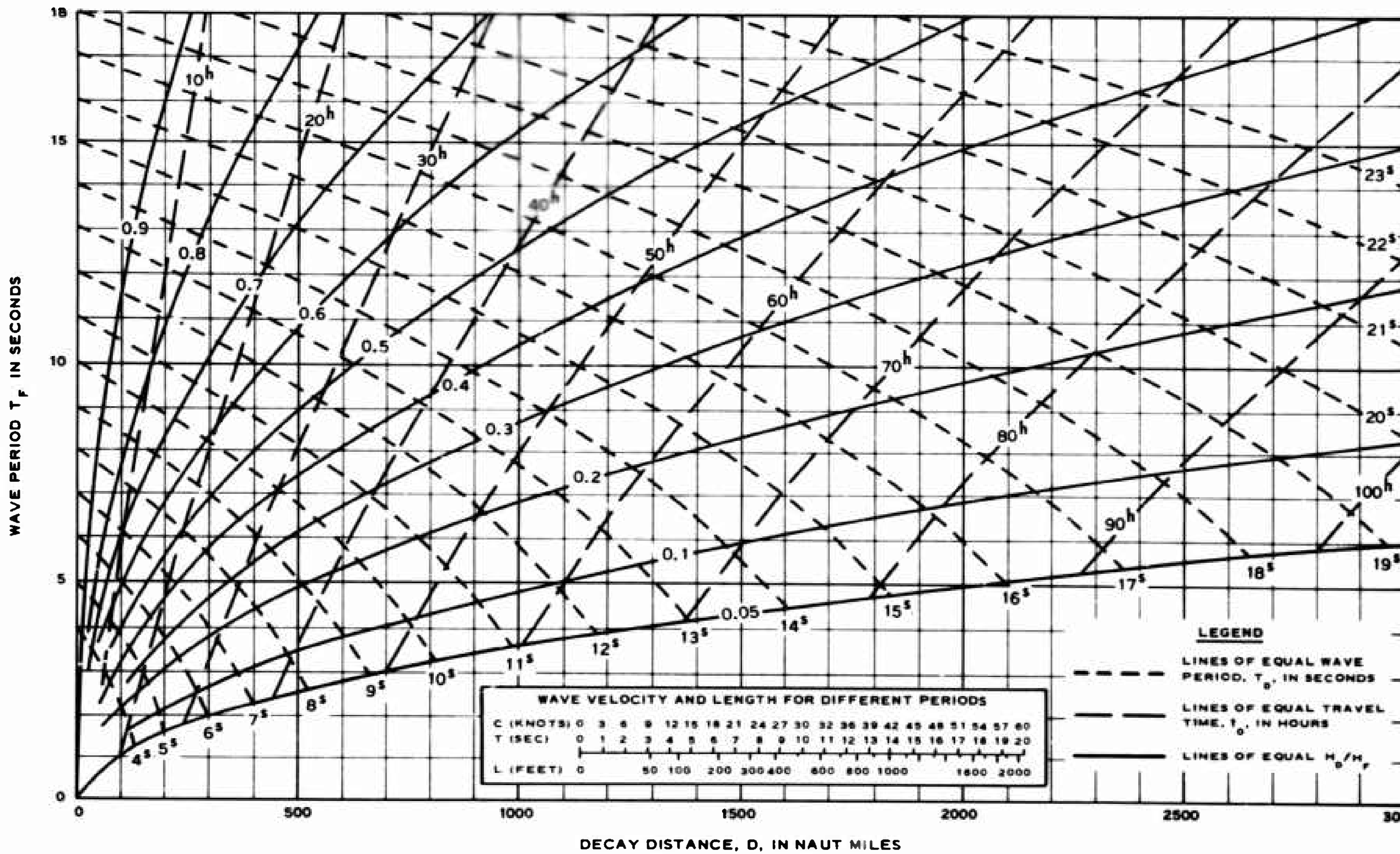
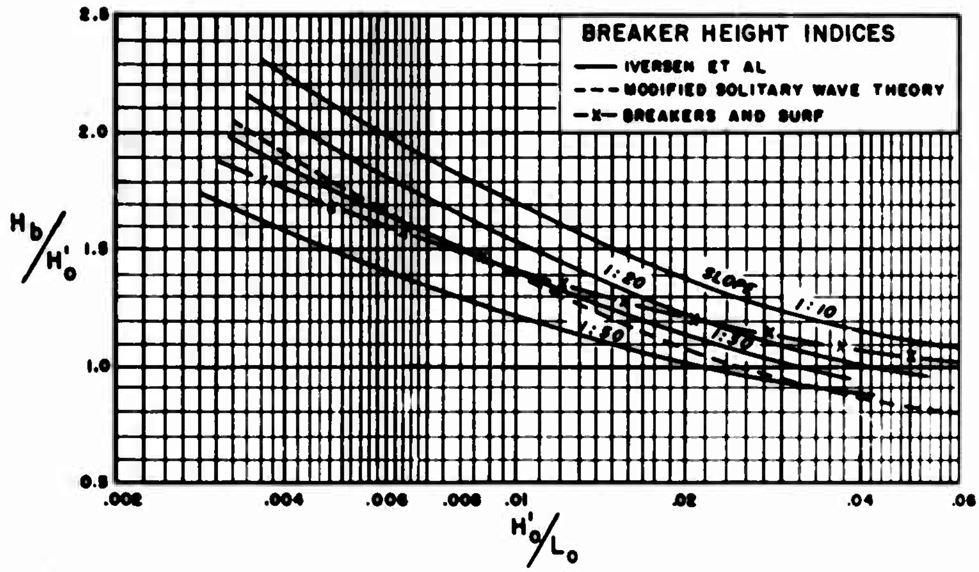
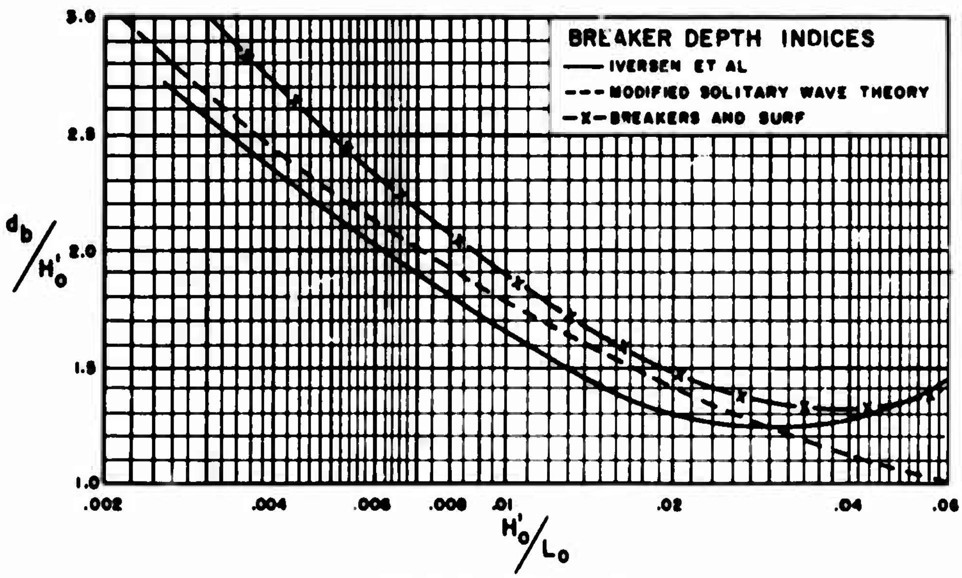


Fig. VI-3. Wave height and period at end of decay distance



BREAKER HEIGHT INDICES



BREAKER HEIGHT INDICES

Fig. VI-4. Height of breakers and breaking depth

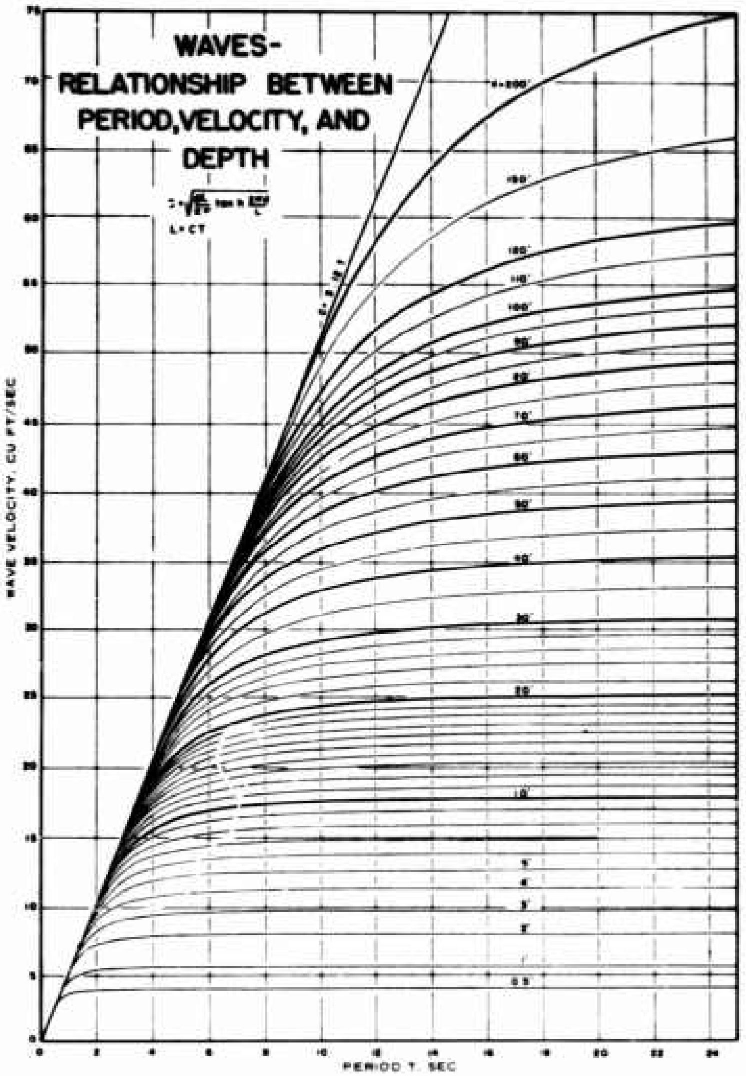
contours, the wave is refracted or turned as it moves into shallow water so as to align itself more nearly parallel to the contours of the beach. This action usually spreads the wave energy over a wider section of the beach and results in a lowering of the wave and breaker height. However, this refraction effect may, in some cases, result in the convergence of concentration of energy against some particular locality on the shore by means of a localized buildup of waves.[3c]

VI-14. Waves approaching a shore may have a portion of their crests interrupted by islands or breakwaters. That portion of the wave crest passing the obstruction to some extent moves in behind the obstruction in an attempt to fill the wave vacuum left by the interrupted portion of the wave. This action, known as wave diffraction, results in some lowering of the height of the uninterrupted portion of wave crest.[3c] Although wave refraction and diffraction are independent phenomena, they may both occur simultaneously.

VI-15. Wave steepness is expressed by the ratio of the wave height to the wave length. This term, generally used to describe this characteristic in deep water since the steepness changes as the wave moves into shallow water, serves somewhat as an index of erosive action to be expected from a wave.

Wave currents

VI-16. The orbital paths of movement of the water particles in an unbroken wave are in effect orbital currents, which, in deep water, are circular in form and in shallow water, before the waves break, are elliptical in form. Near the bottom in shallow water, the ellipses assume a flattened shape so that the water movement is essentially an oscillating horizontal movement. Fig. VI-5 describes a method of approximating the maximum orbital surface velocities which will be somewhat greater than bottom velocities. Movement of the bottom particles results when the critical tractive velocity for the particular bed material involved is reached or exceeded. The bed particles tend to oscillate back and forth with the oscillating wave currents, some particles moving as bed-load creep and some by abrupt irregular advances. Through some mechanism not yet clearly understood, certain waves tend to produce net seaward movements of bed material while other waves tend to effect a net shoreward movement. As a rule, steep waves, i.e. those with large height to length ratios, move material seaward, and flat waves, i.e. those with small height to length ratios, tend to produce a shoreward movement. Thus, the steep waves accompanying a local storm tend to erode material from a beach and, conversely, the long low waves, or swells, reaching the shore from distant storms tend to build up the beach. Also, there is a tendency for the coarser bed particles to be deposited on the beach and the finer particles to be moved seaward. Although these tendencies are generally evident,



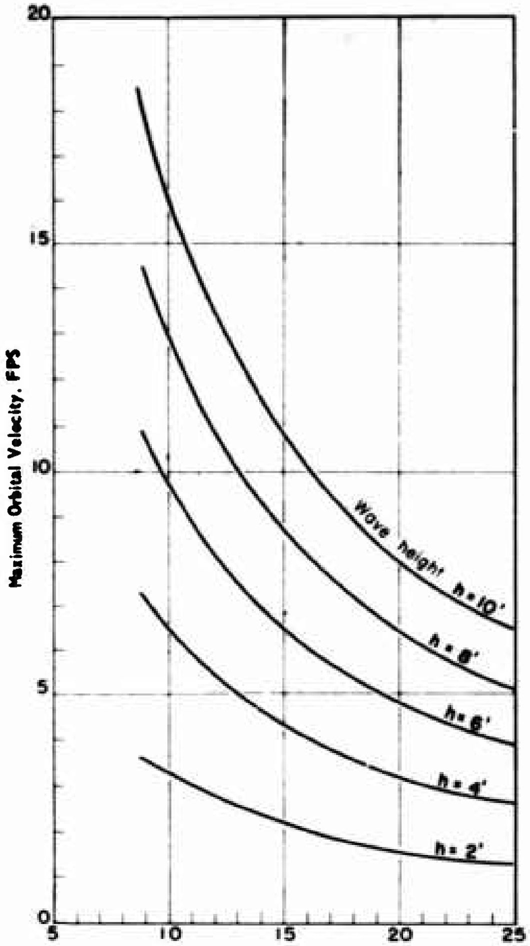


Fig. VI-5. Maximum orbital surface velocities the understanding of their mechanics is still on a qualitative basis. Many beaches undergo fairly well defined seasonal changes, eroding during the winter storm season when waves from local storms control the beach changes and rebuilding during the fair-weather summer season.

VI-17. As the wave breaks upon the beach, the wave becomes essentially a wave of transiation, the orbital motion of the water particles is broken up, and a large part of the wave energy is dissipated in the form of turbulence; this turbulence produces a greatly increased stirring of the bottom materials. The plunging breaker, usually found on beaches with fairly steep bottom slopes (1:25 or steeper) or with a pronounced offshore bar, dissipates its energy very quickly over a relatively short distance. The spilling breaker, usually found on beaches with fairly gentle slopes (1:25 or less), breaks rather gradually and dissipates its energy over a wider stretch of beach. The type of breaker is related to wave steepness as well as to the beach slopes. Breakers found in deep water result from excessive wave steepness and are not to be confused with the two types of shallow water breakers just described.

VI-18. The breaker, or surf, zone is that part of the beach most active insofar as bed movement of materials is concerned, and extends from the point of the breaking wave to the limit of uprush on the beach. The turbulence accompanying both the plunging and spilling breakers increases the amount of bed movement in the breaker zone. The internal velocities produced by a breaking wave can be approximated from the curves shown in fig. VI-6.

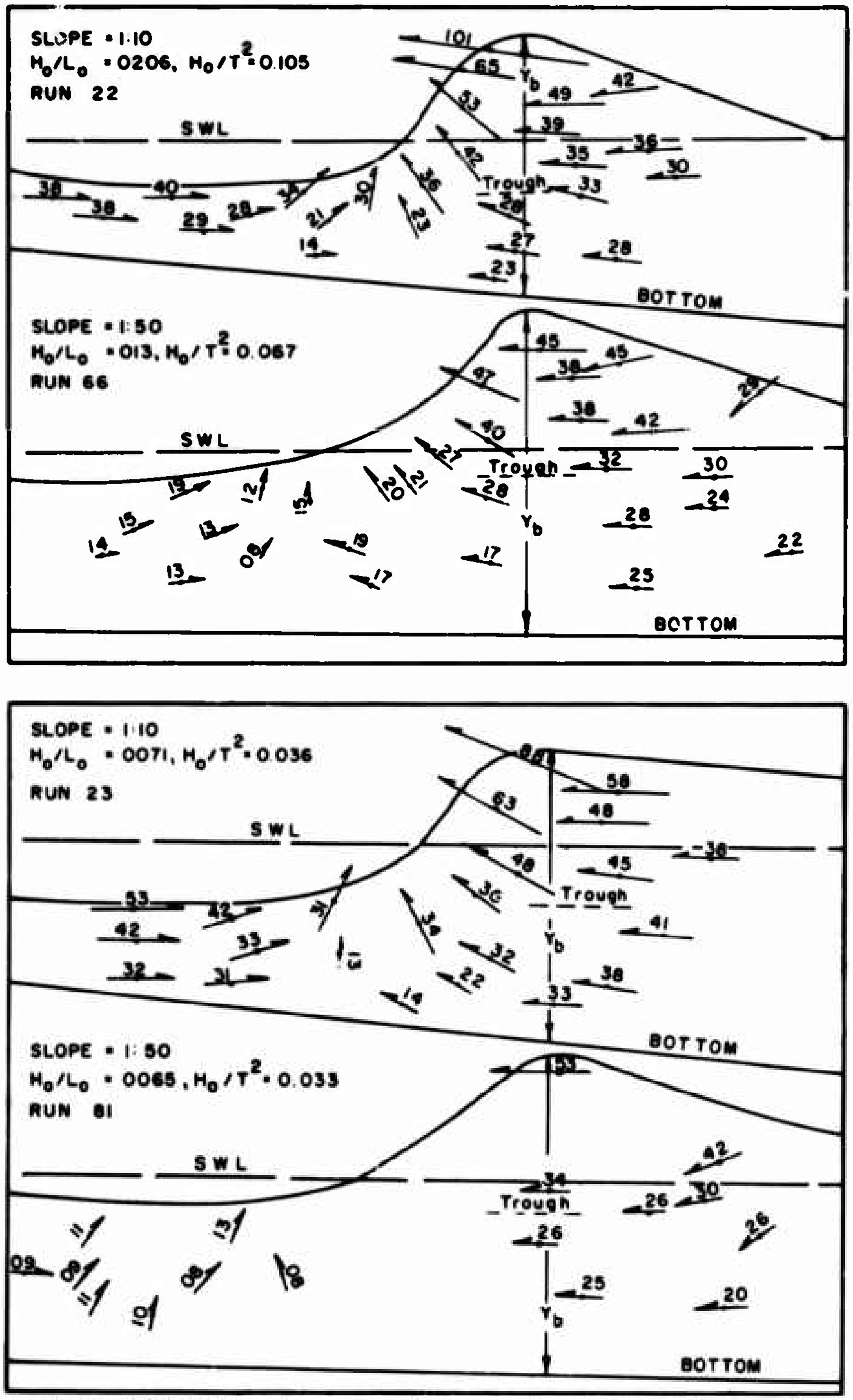
Wave erosion action

VI-19. Waves are responsible for the erosion of land above ordinary mean sea level through two basic actions either individually or in combination. One basic action involves the transportation of beach material from shallow into deeper water by internal wave currents as previously discussed. The other basic action results from littoral currents which will be discussed subsequently in more detail. Wave erosion is most prevalent during periods of excessive local storms such as might accompany a hurricane sweeping up a coast or an unusual rise in water level such as the occasional 2- or 3-yr stands of very high lake levels in the Great Lakes. The early 1950 high levels in the Great Lakes permitted the waves to progressively erode the shore with resulting damages estimated in millions of dollars.

Littoral Processes

Origin of littoral currents

VI-20. The incidence of waves on a beach, as influenced by the hydrography



NOTE: NUMBERS ARE = VELOCITY / $\sqrt{g\,Y_b}$ EQUAL BREAKER HEIGHTS

Sand Transport by Littoral Currents, by J. W. Johnson

Fig. VI-6. Typical velocity fields of waves of various steepnesses breaking on beaches of slopes 1:10 and 1:50

of the shore area, results in the creation of littoral currents. These currents produce mass movements of water in the shore zone superimposed upon the wave pattern and may appear as alongshore currents parallel to the shoreline or as rip currents seaward of the beach.

VI-21. Waves approaching a shore at an angle give an alongshore impulsion to the water masses. For a given wave train, the greater the angle of wave approach, the higher the velocity of the resulting alongshore littoral current. Although alongshore velocities in the order of 10 to 50 ft per min are common, velocities as high as 300 or 400 ft per min have been measured in the surf zone. Wave trains approaching nearly perpendicular to the shore produce little or no alongshore littoral currents of definite direction; if alongshore currents arise from such wave action, they are of low velocity and alternate frequently in direction.[3d]

VI-22. Excess water reaching the shore area through the impulsion of breaking waves sometimes returns to the sea as a band of agitated water known as a rip current. Rip currents are usually of temporary character and prediction of their occurrence, velocity, and duration, on the basis of present knowledge, is uncertain. Although sometimes referred to as "undertow," there is no evidence to indicate that these currents tend to confine their action to the shore bottom. Undoubtedly, rip currents have a significant role in the movement of beach material, but the exact nature of their effect is not fully defined at present.

Littoral drift

VI-23. Littoral currents are responsible for the gross movements of water masses along a coastal shore zone. These alongshore movements of water in combination with the action of internal wave currents which stir the shore bottom and place the bottom particles in motion are the mechanical means for the development of alongshore movements of beach materials, referred to as littoral drift. Littoral drift, which in many instances is the dominant factor in assessing the effects of wave erosion, is subject to reversals of direction depending on changes in the direction of wave approach and the local hydrography. Although there are exceptions, most localities show a dominant resultant littoral drift direction over the years, and determination of this predominant direction is of major importance in the design of shore-protection works.[3e]

Magnitude of littoral forces

VI-24. The magnitude of the littoral forces determines the amount of beach material moved by these forces alongshore each year. This littoral drift movement, usually expressed as a specific number of cubic yards of material passing a selected profile normal to the shore within a given period of time, is in effect the average net alongshore drift of beach material. Although the gross movement, including both upcoast and downcoast, is usually much greater, the net movement is generally used to define the rate of littoral drift. Reliable net rates of littoral drift are determinable only where the drift has been trapped for a number of years by the construction of a jetty, breakwater, or other structure that is a substantially complete littoral barrier. Net rates of littoral drift up to in excess of 500,000 cu yd per yr have been measured along the ocean shore of the United States with rates of 300,000 cu yd per yr being fairly common. Net rates on the Great Lakes are usually much lower, being in the order of 25,000 to 100,000 cu yd per yr.[3f]

Predominance of direction

VI-25. As previously mentioned the direction of littoral drift movement is generally determined by the direction from which waves approach a shore. Accordingly, the predominant direction of littoral drift in an area is, as a rule, closely related to the predominant ocean wave pattern in the area. On some beaches, the ocean wave pattern varies in such a manner throughout the year as to produce very little net movement of littoral drift, resulting in very stable conditions where drift leaving the beach on the downdrift side is very nearly replaced or balanced by new material moving in from the updrift direction. Although there are exceptions, erosion due to an imbalance in the quantity of littoral drift entering and leaving an area is normally a slow process on a shore that has had time to adjust itself to the prevailing conditions. The sudden imposition of an obstruction or barrier to the drift movement may, however, bring about rapid erosion on the downdrift side of the structure and equally as rapid accretion of materials on the updrift side.

Means of measuring littoral movements

VI-26. Due to incomplete knowledge of all factors contributing to the direction and magnitude of littoral movements, and the physical difficulties associated with full appraisal of the complex interrelated forces at work in offshore areas, it is presently possible to only estimate the full characteristics of littoral action in any particular locality. As may be noted from the preceding discussion, examination of the location of eroded or accreted areas adjacent to a structure constructed so as to obstruct the alongshore movement of drift materials will reveal definite evidence of the predominant direction of littoral drift. Also, study of condition surveys or aerial photographs taken over a number of years will provide further evidence of the predominant littoral direction. Establishment of beach profiles normal to the shoreline and observations of depths along these profiles over a period of time will show evidence whether or not the beach in the profile area is being eroded or built up by movement of beach materials. Measurement of the quantities of materials deposited on the updrift side of an obstructing jetty, breakwater, or other structure will provide some indication of the quantities of materials being moved by the portion of the littoral drift movement thus interrupted.

Characteristics of littoral action

VI-27. There is a tendency for dredged entrance channels to shoal through the trapping of littoral drift material and this tendency to trap material appears to become more pronounced when the dredged depths materially exceed the natural entrance depths. Entrance channels without some means of protection such as offered by jetties may trap the gross littoral drift movement of material from both the upcoast and downcoast directions if the channel is continually maintained below natural depths by dredging only. Hence, along coastal areas where the littoral movement is significant, it becomes apparent that some practical means of preventing the deposition of littoral materials in dredged entrance channels can become an important consideration in the proper design of such entrances.

Shoaling in Ocean Entrances

VI-28. Shoaling in ocean entrances is due to a combination of factors related to environmental land conditions as influenced by certain ocean phenomena. Environmental land conditions contributing to the magnitude and location of shoaling within the entrance are the geographic characteristics of the estuary, inlet, or lagoon as related to its size and shape; the amount of freshwater discharge as compared to the tidal prism; and the amount of sediment, both dissolved and suspended, carried by streams tributary to the entrance. Ocean phenomena influencing shoaling in an entrance are the prevailing wave pattern, wave-generated currents, littoral processes, and density currents.

VI-29. The broad process of shoaling at estuarine entrances may be described generally by the following sequences:

a. Littoral drift material moves into an entrance under the impulse of ocean wave action and begins the formation of a shoal.

b. This shoaling continues until the entrance has been sufficiently constricted to cause tidal currents to increase to the point where shoal materials are swept back and forth by the ebb and flood currents of the tide.

c. The shoal is then molded by the interaction of the waves and currents in an attempt to reach a condition of equilibrium.

d. The addition of material to the shoal by wave action bringing in littoral drift and in some cases by freshwater flows bringing material down the estuary tends to enlarge the shoal. This enlargement temporarily upsets the conditions of equilibrium at the entrance.

e. The net result of the above is, in many cases, a constant shifting in the location and depth of the more or less well-defined channel across the bar or shoal. These changes are, of course, hazards to navigation.

VI-30. At inlets, where the length is only a few times greater than the width, the shoal material in transit through and around the inlet is primarily that brought into the inlet by littoral movement along the shore face under the influence of ocean waves. The action of the littoral drift and the tidal flow in the inlet generally results in the creation of an outer bar at the ocean mouth of the inlet, an inner bar (or middle ground shoal) at the bay mouth of the inlet, and a relatively deep gorge section between the two bars. Since most inlets are, in effect, short canals through a low, sandy, barrier ridge or island, the inlet beds are composed of relatively unconsolidated materials and are therefore free to migrate under the pressure of littoral processes. Such migratory characteristics are most common where the littoral drift is predominantly from one direction. East Rockaway Inlet, located on the south shore of Long Island and shown in fig. VI-7, is an example of a migrating inlet; this inlet migrated westward some 4 miles in the 92-yr period 1835-1927.

VI-31. The size of an inlet may vary considerably from year to year depending upon the types and frequencies of prevailing storms in the area. Some inlets are created by severe storms and, conversely, some inlets are subject to closure when storm waves move an unusually large amount of littoral material into the inlet mouth. Once closed, inlets may stay closed indefinitely. The tendency to close is particularly noticeable where a number of inlets feed into the same bay or lagoon. Several inlets to Pamlico Sound, North Carolina (fig. VI-8), have a history of opening and closing over the past 200 years.

VI-32. Since the lagoon area immediately inside an inlet is usually of the same or approximately the same degree of salinity as the adjacent ocean waters, the influence of density currents is generally of minor importance in the formation of inlet shoals. The mechanism of transport of material brought within the influence of the inlet by littoral drift is fairly evident. Tidal currents pump the material back and forth through the inlet channel. Some of this material finds its way onto the outer bar and some of the material may be deposited on the inner bar. The outer bar material continues to be subject to the entrance tidal currents and to the littoral forces, while the inner bar material is subject to these same tidal currents and also the smaller local wave action generated over the lagoon. Further, the inlet shoulder on the side fed by the dominant littoral drift is

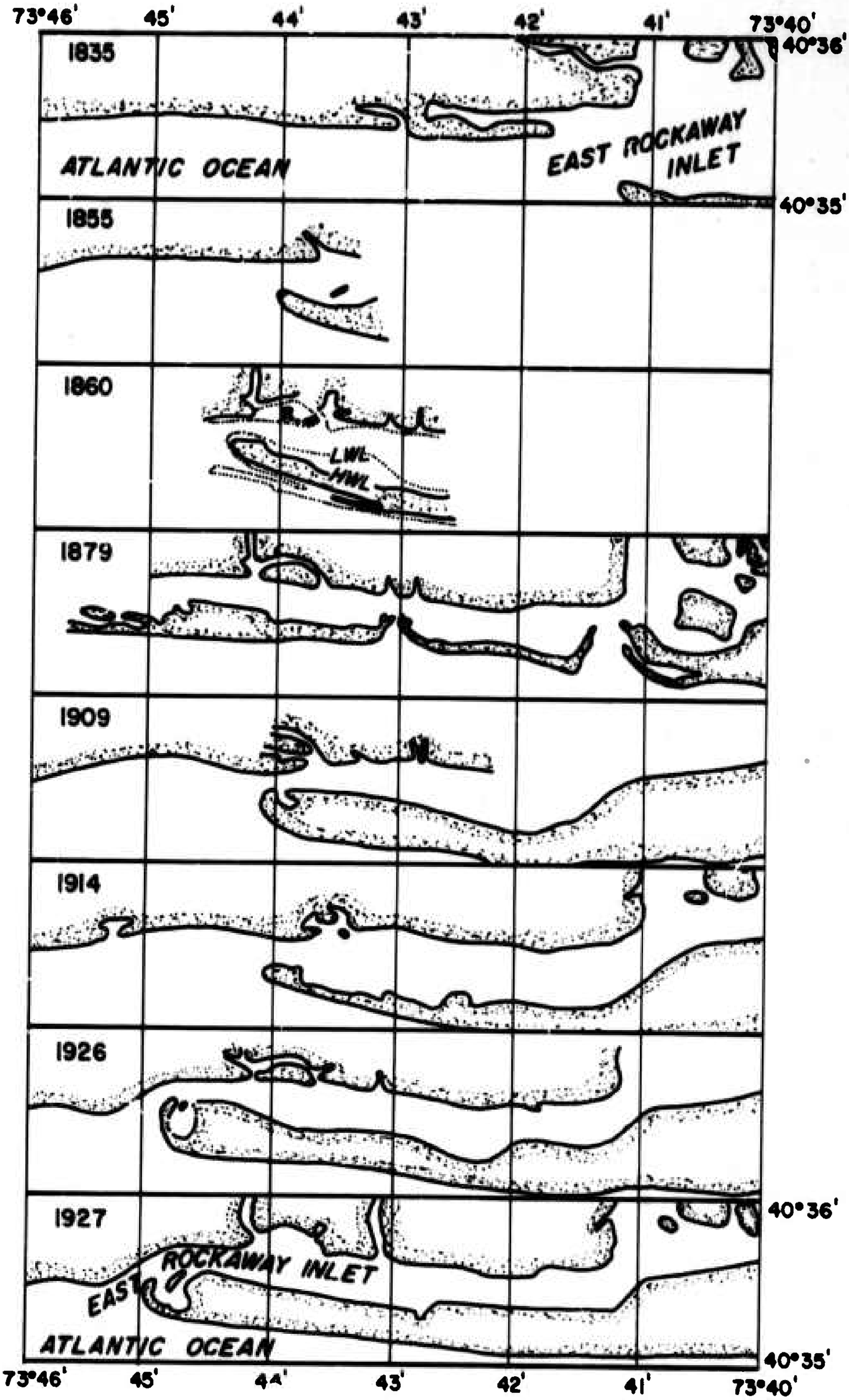
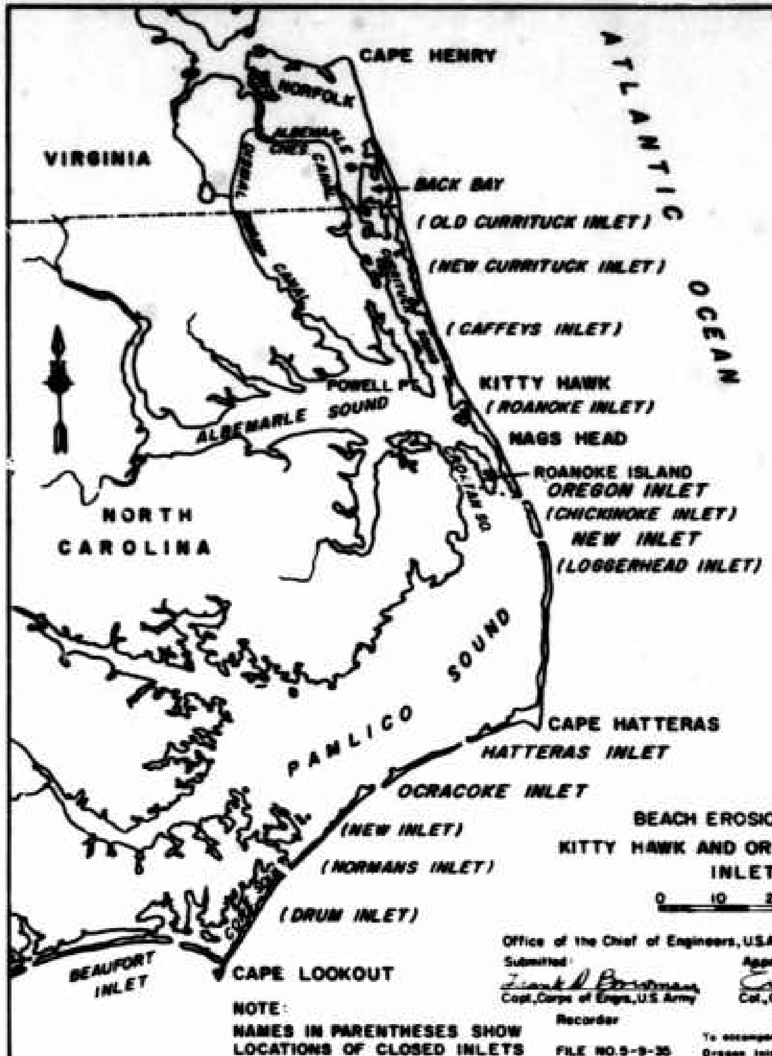


Fig. VI-7. Shoreline changes, East Rockaway Inlet, New York



COMPARATIVE CONDITIONS OF INLETS ON NORTH CAROLINA COAST

NAME OF INLET	CONDITIONS AS SHOWN BY MAP OF - Hariot, 1585	Lawson, 1708	Wimble, 1738	Mouzin, 1775	Atlantic Neptune, 1780	Lewis, 1795	U.S. Coast and Geodetic Survey, 1875	1934	REMARKS
Currituck	O	O	O	C	C	C	C	C	
New Currituck	C	–	C	O	O	O	C	C	Closed in 1828
Caffeys	O	–	C	C	C	O	C	C	Closed in 1800
Roanoke	O	O	O	O	O	O	C	C	Closed in 1798
Oregon	O	–	O	O	O	O	O	O	
New (Dare County)	C	–	C	C	C	C	O	O	New Inlet (Dare County) closed in January 1922. An unsuccessful attempt was made to reopen it artificially in 1924. The inlet reopened during a storm in March 1932
Loggerhead	C	–	C	C	C	C	O	C	
Chickinoke	C	–	O	O	O	O	C	C	
Hatteras	O	O	O	O	O	O	O	O	
Ocracoke	O	O	O	O	O	O	O	O	Known as "Wokoken" in 1585
New	O	–	O	C	–	C	O	C	
Normans	O	–	O	C	–	C	C	C	
Drum	O	–	O	C	–	C	C	C	
Beaufort	–	O	O	O	–	O	O	O	Known as "Core Sound" in 1708, 1738 and 1775

(O) Indicates open at time; (C) Closed; (–) Not given

BEACH EROSION BOARD
KITTY HAWK AND OREGON INLET, N.C.
INLETS

0 10 20 30 MI.

Office of the Chief of Engineers, U.S.A. Washington D.C. Sept 27, 1934

Submitted: Frank A. Bowman, Capt., Corps of Engrs., U.S. Army, Recorder

Approved: Ernest A. Graves, Col., Corps of Engrs., U.S. Army, Senior Member

To accompany report on Kitty Hawk and Oregon Inlet, N.C. dated Feb. 15, 1935

FILE NO. 5-9-35

NOTE:
NAMES IN PARENTHESES SHOW LOCATIONS OF CLOSED INLETS

Fig. VI-8. Pamlico Sound, North Carolina reinforced more rapidly than the downdrift shoulder. This action tends to crowd the inlet and cause it to migrate in the direction of dominant littoral current. Although the mechanism thus described may appear relatively simple, a reliable prediction as to the future behavior of an inlet is extremely difficult. Possibly, the most indeterminate factor in attempting a prediction of future action is the uncertainty of the magnitude, direction, and frequency of the storms striking the area. A severe hurricane may temporarily upset a well-established pattern of development by closing an inlet or, by reason of high tides and winds, break across a barrier beach and open a new inlet.

VI-33. The sediments in a lagoon receiving upland discharge are principally those brought in from the ocean shore by tidal and littoral action through the inlet and those brought down by tributary streams. Wave wash over low-barrier beaches can move appreciable amounts of material into a bay under certain conditions, as can wind blowing over unvegetated dunes and sand ridges. The former sources are usually the most significant. Generally, the forces tending to bring material into a bay or lagoon are far greater than the forces tending to remove the material. Thus, the lagoon approaches the nature of a sediment trap, and consequently, most lagoons are in a deteriorating condition.

Corrective Measures

Criteria requirements

VI-34. The magnitude and direction of littoral drift past a navigation entrance channel into an estuary or a lagoon together with the demands of navigation establish the criteria requirements to be met by any effective corrective measure or measures. In establishing these criteria, consideration should be given to the optimum balance between erosion and shoaling to be anticipated from prevailing littoral movements. This, of course, should recognize that any planned interruption of natural littoral movements will result in deposition of littoral materials updrift from the obstructing structure and either produce erosion or accelerate existing erosion of the downdrift shoreline due to the cutting off of littoral movements. Another consideration, which sometimes is not given proper weight due to lack of sufficient field data, is a realistic appraisal of the effectiveness of possible corrective measures. Lastly, but not of least importance, is the requirement that the cost of the corrective measure selected be justified by the benefits which may reasonably be expected to accrue. Although in the design of modern entrance channels it still is necessary, in the absence of full knowledge of prevailing conditions, sometimes to depend upon judgment considerations, this dependency is becoming less and less necessary to practical entrance channel planning, design, and development as knowledge of tidal hydraulic phenomena advances. The ability to ascertain the economic feasibility of any proposed entrance improvements progressively becomes more firm with each advance in technical understanding of tidal hydraulic phenomena.

Establishment of desired balance between erosion and shoaling

VI-35. As mentioned in the preceding paragraph, the design of any corrective measure should realistically appraise the possible effect of that measure on the natural balance of the forces of erosion and shoaling prevailing in the entrance region. These forces, established by nature along every coastal area, regulate the stability of the shoreline through definite patterns of erosion and shoaling which are modified only by longtime changes in oceanographic and hydrographic environment. To ignore these forces and their effects or to fail to design the corrective measures to work in conjunction with, instead of contrary to, these controlling forces is to invite inordinate heavy future maintenance costs as well as to create a continuing hazard to navigation.

VI-36. Basic to the design of any effective corrective measure is a determination of the magnitude and predominant direction of littoral movements. Once these facts have been reasonably ascertained, it will be possible to appraise in a general manner the useful life of a jetty or similar structure designed to preclude or alter the movement of littoral drift materials into an entrance channel. It will also be possible to determine the adverse effects of such structures by interruption of the littoral material movements to downdrift shore areas. At estuaries where the discharge of fresh water is significant, the problem shoaling in the entrance channel is made more complex by density current action.

VI-37. The design of any structure interrupting the natural movement of littoral materials should give appropriate consideration to minimizing the effect of that structure on the littoral movement. This may be accomplished by developing the means whereby littoral materials deposited behind the structure barrier may be periodically removed in order to ensure the continual retention capacity of the structure and thus its effectiveness in halting the movements of littoral material into the dredged channel. Also, some means should be developed whereby movements of littoral materials along downdrift shore areas are restored and maintained to ensure the continued stability of these areas.

Types of corrective measures

VI-38. Shorelines may be made more stable by the application of shoreline fixation measures to reduce the effects of wave erosion and thus, indirectly, retard the amount of materials placed in motion by wave action and littoral forces.

Groins, artificial beach fill, and sand bypassing plants have been used to influence the pattern of littoral drift movement or to assist in restoring areas eroded by littoral drift action. Although designed primarily to reduce shoaling of navigation channels, jetties often act as catchment structures to trap and retain littoral drift intercepted by these structures. Breakwaters, designed to provide protection against wave action, may act in a similar manner.

VI-39. Shoreline fixation involves the construction of a shore structure designed to withstand wave attack and to prevent further encroachment of the waves into the shore. Seawalls, bulkheads, or revetments of concrete, steel, wood, or stone are generally used for this purpose. Although these structures do not rebuild a beach, they are usually effective in preventing encroachment of the sea beyond a selected fixed line.[3g] In emergencies, sandbags, brush, and wire fencing are sometimes used as temporary protection measures.

VI-40. Groins are usually timber, concrete, or stone structures extending seaward of the high-water line for the purpose of decreasing the rate of littoral drift leaving an area. Their function in an ocean entrance problem is to reduce the quantity of material reaching the entrance, or to reduce erosion downdrift of an entrance. Groins are usually constructed in groups of two or more so as to extend their protection over a given section of the shoreline. If an appreciable net littoral drift movement passes the groin area, these structures may entrap materials and thereby build up the shoreline in an updrift direction. Should, however, the net littoral drift movement be small, groins may serve to stabilize the shore or greatly decrease the rate of erosion. Although groins may sometimes thus produce an accreting shoreline updrift of the structures, they almost always cause some downdrift erosion due to interception of the littoral drift movement or by deflection of this movement seaward so that its effect as a stabilizing factor on the downdrift shore is lost. Groins are not effective against forces tending to move materials from the beach to deep water. They are best adapted as measures for retardation of erosion due to alongshore littoral currents.[3h]

VI-41. Artificial beach fill has been used successfully to replace beach materials which have been lost by erosion. The function of this measure in solving ocean entrance problems is to offset erosion downdrift of the entrance. Ordinarily, the new fill material should be at least as coarse, and preferably coarser, than the materials it replaces. As may be expected, new beach fill must be replenished periodically as the fill itself provides no means for combating the wearing effect of the attacking forces. Where the new fill is subject to severe erosion by alongshore littoral currents, a combination of beach fill and groins will sometimes prove to be the most feasible means of protection.[3i] Beach fill has an added advantage of providing a beach suitable for recreational purposes.

VI-42. In instances where the littoral drift movement has been intercepted by the construction of a jetty or breakwater, one generally successful method of restoring the littoral action to downdrift shores involves the pumping of impounded sand past the obstructing jetty or breakwater. Such steps have been successfully employed at Santa Barbara and Pt. Hueneme, California, and at South Lake Worth Inlet, Florida.

VI-43. At Santa Barbara, California,[4] a detached breakwater, constructed during 1927-28 to provide a small-boat basin and subsequently in 1930 extended to the shore (see fig. VI-9), intercepts a strong littoral drift movement from the west. By 1937, the shoreline immediately west of the basin accreted materially resulting in the seaward movement of the mean lower low waterline a maximum of about 800 ft. Interruption of the littoral drift in this instance resulted in substantial deposits of littoral-borne materials in the basin estimated to average about 275,000 cu yd annually. Downdrift to the east the shoreline, being starved of littoral drift supply, eroded severely. In an attempt to restore these eroded beaches, 200,000 cu yd of material dredged from the basin was dumped in 22 ft of water approximately one mile east of the breakwater and 1000 ft from shore, forming a mound 2200 ft long and 5 ft high. It was hoped that the waves would move this material onshore and eastward. This, however, did not occur, and as a consequence, dredged material was thereafter deposited by pipeline on "feeder beaches" just east of the breakwater which expedient proved more successful.

VI-44. The authorized project for Lake Worth Inlet, Florida,[5] consists of an entrance channel 35 ft deep by 400 ft wide merging with an inner channel 33 ft deep by 300 ft wide, a turning basin, bank revetment, and jetties. The jetties were constructed and the channels dredged to lesser dimensions under prior authorizations. Work under the latest authorization is underway. The jetties intercept a predominantly southbound littoral movement of materials resulting in a very substantial buildup of the shoreline north of the inlet and erosion of the beaches to the south. Restoration of these starved beaches by means of groins was attempted, but on the whole proved unsatisfactory. As an alternative to groins, a stockpile of sand containing about 280,000 cu yd was pumped from Lake Worth in 1944 onto the beach at a point about 1500 ft south of the inlet. Observations four years later disclosed that the stockpile, distributed by the surf, had benefited the beach for one-quarter of a mile to the north and for a mile and one-half to the south. Pursuant to the recommendation of the Beach Erosion Board, some 2,250,000 cu yd of sand was deposited in 1948 and 1949 on the beach in five stockpiles spaced some 4000 to 6000 ft apart, the northernmost stockpile being about 9000 ft south of the inlet. In 1953, it was observed that, while most of the

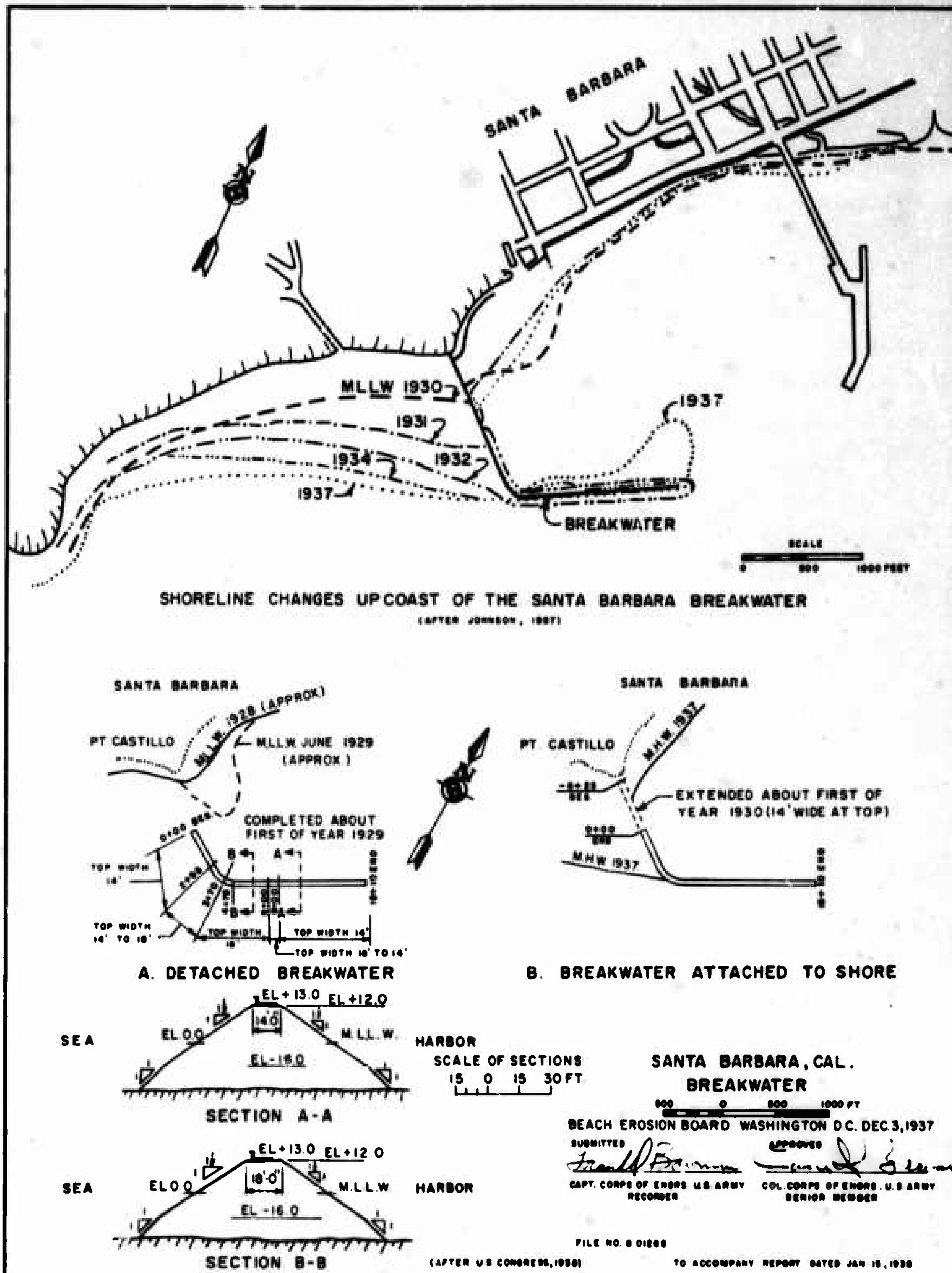


Fig. VI-9. Santa Barbara Harbor, California stockpiled material had moved southward and had benefited the shore for a distance of five miles or more south of the inlet, the beach for about ten miles south of the inlet still needed nourishment. After considering several possible alternative means of nourishing of south beaches in the light of the favorable experience gained from the use of stockpiles, it was decided to install a permanent sand bypassing plant near the end of the north jetty at Lake Worth Inlet. This plant, containing dredge pump equipment, removes material from an area just north of the jetty by suction line from which it is pumped back through a discharge line along the jetty to cross the inlet in a trench 3 ft below the bed of the inlet channel and discharge at the south face of the south jetty (see fig. VI-10). It is anticipated that some 150,000 cu yd of sand will reach this bypassing plant annually and that, of that total, the plant will be able to transfer about 100,000 cu yd to the south beach. Several years of observation will precede definite appraisal of the merits of this sand bypassing plant.

Selection of Measures

VI-45. The selection of the most effective plan for the prevention of wave erosion or the best plan to minimize the effects of littoral action on navigation entrance channels consistent with the economics of a given situation is generally a rather involved problem. Frequently, it will be found that existing information is insufficient to permit a definite determination of either the rate or direction of the prevailing littoral drift and that considerable study will be required to establish those basic facts. Once these boundaries are established, however, application of increasing knowledge of tidal hydraulics will provide a sound approach to a problem's solution. In complex problems, a model study is usually needed to supplement or confirm the conclusions reached by analytical procedures.

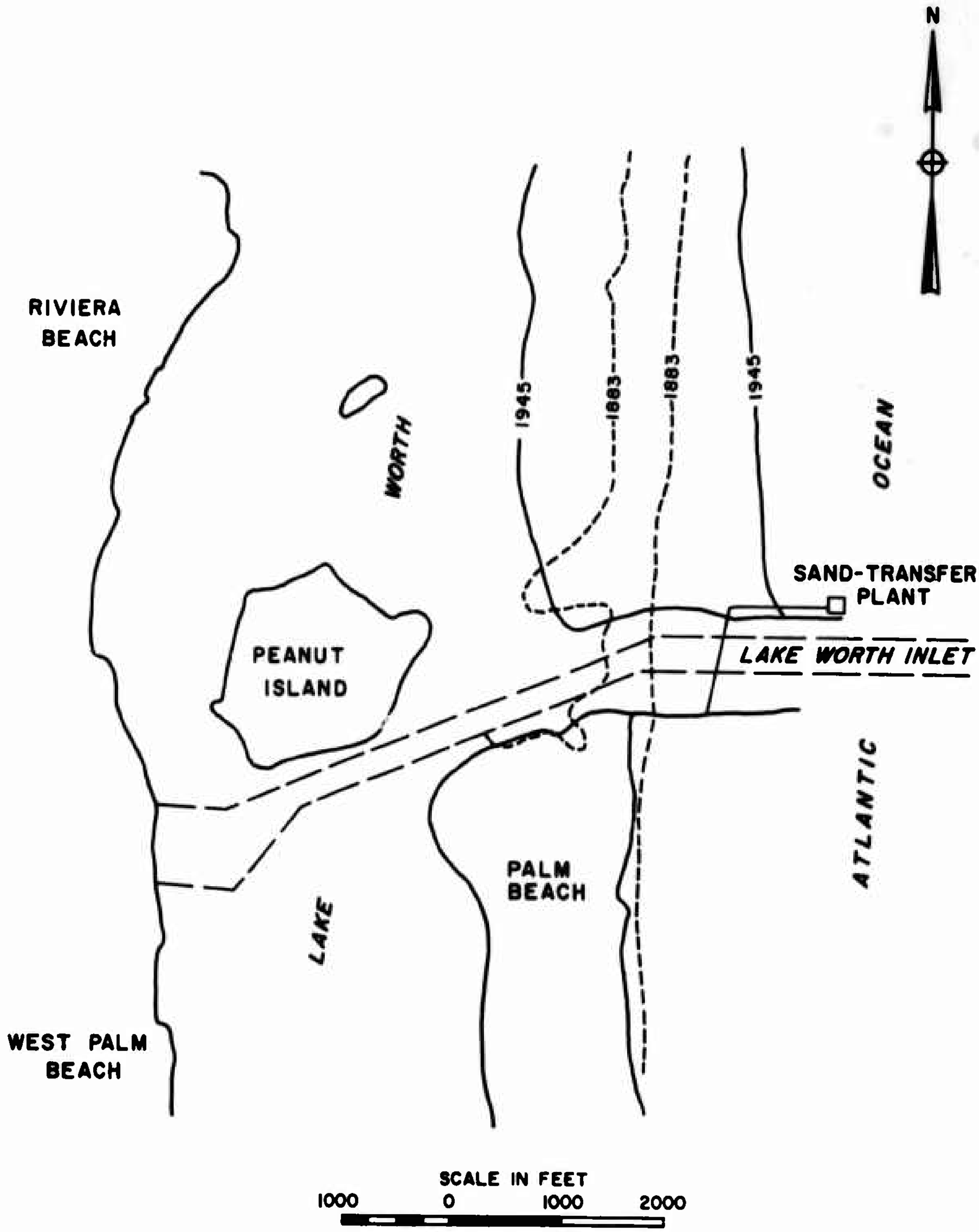


Fig. VI-10. Lake Worth Inlet, Florida

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CHAPTER VII

DREDGING AND DISPOSAL PRACTICES IN ESTUARIES

by

J. R. Johnston and J. C. Marcroft

Introduction

Purpose

VII-1. Waterways which were adequate to accommodate the vessels of earlier years are becoming increasingly inadequate to accommodate present-day vessels, and these cannot be regarded as the ultimate because even larger and faster vessels are being designed. The ever-increasing draft of vessels has imposed a need for greater depths in the entrances and inner channels of our waterways; naturally deep channels are being dredged deeper for the first time, and previously dredged channels are being progressively deepened. Such deepening is termed "new work," but most of such improvements are not permanent, as they must be redredged repeatedly ("maintenance" work) to keep these channels at the depths originally intended. To extend the interval between recurrent maintenance dredging operations, the restored depth may be carried deeper than normal to provide a predetermined amount of catchment capacity below design depth. This excess depth may be termed "advance maintenance," but from a dredging standpoint it is new work the first time it is performed, as it requires the removal of the undisturbed native materials rather than accumulated sediments.

VII-2. Dredging is performed to provide channel access to the inner harbors, and to provide passing basins, turning basins, anchorage areas, slips for between-pier access, and other special purpose requirements. It can often be coordinated with land reclamation, creating areas upon which harbor or industrial facilities will be built, but, of course, dredging may be done with the single-purpose objective of land reclamation or for the building of dikes and other coastal protection works. Dredging also may have as an incidental or primary objective work not related to channel deepening, e.g. replenishment of beaches which have been eroded. A specialized form of dredging for this purpose is known as "bypassing," and is discussed in Chapter VI.

Material

VII-3. Because of the nature of an estuary, much of its bottom is covered with muds or sands, and these will be encountered in almost every job of dredging. However, new work dredging may be in clay strata or in rock of almost any character. Dredging in tough clay will require one of the types of dredges which has a positive cutting action. Rock removal, except for soft or fractured rock, will require blasting (not considered in this report) before it can be removed by any of the conventional types of dredges. Maintenance dredging, on the other hand, will be in unconsolidated material that may even be found to be so soft as to constitute a problem from both the dredging and the spoil-retention standpoints. In one case observed, the return flow from a spoil area had a sediment content of over 100,000 ppm. The material being dredged had a density of only 67.8 lb per cu ft (1085 grams per liter), and 95 percent (by weight) was finer than No. 200 mesh in size. In addition to soft mud, a layer of "fluff" is found lying along the bottom in many estuary channels. This fluff layer, which is often 3 to 6 ft in thickness, also presents a dredging problem.

Characteristics of the estuary

VII-4. The important characteristics of an estuary from the standpoint of dredging are the exposure of the areas to be dredged to severe wave action and strong currents, the existence of tidal bores, traffic conditions, and the availability of disposal areas. The most critical part of an estuary, from the standpoint of maintaining dredged depths, is often at the entrance. This may be the section between the headlands or the section over the outer bar, or both. An unstable entrance may present a costly maintenance problem. Usually, this is the section of the channel experiencing frequent and severe wave action, and suitable disposal areas are often at a considerable distance. However, somewhat less severe but nevertheless disruptive wave action may be experienced elsewhere. Estuaries are often so wide that transverse or oblique winds cause severe wave action, or they may have long reaches where strong winds paralleling the reach can generate large waves. The disposal area problem is, of course, associated with the characteristics of the estuary. Where the shoreline is remote and disposal in the waterway itself will contribute to shoaling of the channel, the dredged material must be transported to a place where it will do no harm. Similarly, highly developed shorelines or valuable wildlife lands may not be used for disposal except at often prohibitive cost. Even in cases where it may be proper from an engineering standpoint to dispose of the dredged material in the waterway itself, this will not be feasible where the adjoining shoreline is highly developed or where the bed of the waterway is a valuable shellfish-producing resource. Obviously, obstructing the approaches to landing facilities thwarts the basic purpose of the navigation channel, and depositing material on a shellfish bed will kill the shellfish.

VII-5. Strong currents may be encountered. These cause difficulties in keeping the dredge in place or in maneuvering. Under some circumstances, the tidal undulation advances swiftly up the estuary with an abrupt rise of several feet, a phenomenon known as the tidal "bore." The sudden surge can capsize dredges of the pipeline, dipper, or bucket types, and almost certainly break the pipeline. It will even cause difficulty for an unwary operator of hopper dredges. Heavy traffic may cause so much disruption of dredging operations of pipeline, dipper, or bucket dredges as to render them almost useless for harbor dredging, unless, in the case of a pipeline dredge, a submerged pipeline is employed.

Dredging Equipment

Dredges

VII-6. This chapter is not intended to be a reference for detailed data on different types of dredging plants, or for the design of dredging equipment. For such data, the reader should refer to some of the many books and articles which have been published on this subject. However, there is presented here a very brief outline of the peculiar capabilities, advantages, and disadvantages of the principal types of dredges which should be considered with regard to the subject of dredging in estuaries.

VII-7. The <u>dipper dredge</u> may be likened to a power shovel mounted on a barge. The hull is spud-mounted to keep the dredge from pitching in rolling swells and to afford stability under conditions of heavy digging. The buckets range in capacity from about 3 cu yd up to about 10 cu yd and are capable of heavy digging up to and including blasted rock. They must discharge their load within the reach of their dipper stick.

VII-8. The <u>ladder dredge</u> is similar to the much smaller land plant used for trench excavation in that it consists principally of an endless chain of buckets which dig and bring the material to the surface where it is dumped into chutes or onto belts and is discharged over the side of the hull. This equipment also operates on spuds to prevent damage from pitching and to give stability for digging. It is capable of heavy digging in blasted rock. While used for special purposes in the United States, its principal use for estuarial dredging is in Europe.

VII-9. The bucket dredge, or "grapple" dredge, is a barge-mounted clamshell, or orange-peel, excavator. It is effective in moderately stiff digging, and is particularly adapted to working in confined areas, such as between piers. The largest of these has an 8-cu-yd bucket and a 240-ft boom. Dredged material must be dumped within the reach of the boom. Some dredges of this type operate on spuds and some operate only on anchor lines. When operating on anchor lines, they have an advantage over most types of dredging plants in that they can be operated in moderate swells in exposed situations.

VII-10. The pipeline dredge consists of a barge-mounted centrifugal dredge pump with the suction line extending beyond the bow and lowered to the bottom by an "A" frame and ladder, and with the discharge line extending beyond the stern and continuing to the selected point of discharge. Some pipeline dredges take material through a plain dredge shoe on the end of the suction pipe, or in some cases a "dustpan" shaped head with water jets is added. Other pipeline dredges are equipped with a rotating cutterhead at the end of the suction pipe to dig the material for ready removal. Sweeping the suction pipe over the area at constant depth results in an operation which achieves a uniform depth free from high spots. This type of dredge can be operated safely only in the absence of adverse swells, but it is nevertheless the most versatile and widely used dredge. It is made in sizes (pipeline) from 6 in. to 36 in. and the more powerful can dredge without blasting in rock of #3 hardness on Mohs' scale (#1-talc to #10-diamond). A 30 in. dredge may have 8000 hp on the dredge pump and 2500 hp on the cutterhead, and under a favorable dredging situation can dredge 4500 cu yd per hr on pipeline lengths of 15,000 ft. The rate of dredging will decrease with increase in difficulty of digging, increase in length of discharge pipeline, and increase in lift to discharge elevation.

VII-11. The hopper dredge is a self-propelled vessel containing a centrifugal dredge pump, the suction of which extends through the side of the hull and extends as a "drag pipe" to a terminal "drag head" which is lowered to the the bottom and sucks material as the vessel proceeds on course. The pump discharge is through gates and sluiceways to the midships hoppers where the solids settle in the hoppers and the supernatant water spills over weirs and scuppers to return to the sea. The loaded dredge proceeds to the disposal area where the spoil is discharged through gates in the bottom of the bins. Recent developments have added pumping discharge capabilities to hopper dredges. This type of dredge is an entirely self-contained unit, can operate in fair-sized swells, and can dredge at sites far removed from the point of spoil disposal. For these reasons this is a very versatile type of equipment.

Transport

VII-12. A dredging plant is identified also by its characteristic form of material transport, from dredged site to disposal site. The dipper dredge, ladder dredge, and bucket dredge each must discharge its dredged material close alongside its hull. This requires that each such operation be serviced by a fleet of dump scows and attendant tugs, and for economical operation, that the capability of this equipment be at least equal to the capability of the dredging equipment. Such a determination would be based upon considerations of rate of dredge output, capacity of scows, distance to dumping ground, speed of tow, and delaying effects of weather and other vessel traffic. Scows range in capacity from 500- to 3000-cu-yd "scow measure," which usually is about ten percent greater than "place measure." This form of disposal ordinarily does not contemplate beneficial use of the dredged material.

VII-13. Disposal in the case of pipeline dredging consists of extending the discharge pipeline to the point of desired disposal. A minimum length of flexible-joint, floating pipeline must extend from the dredge to permit the dredge to swing and advance in the cut. In situations where the traffic is heavy, it may be necessary to install an underwater line across the channel. While pipeline method of spoil transport has important advantages over other methods, it has disadvantages also, one of which is that many situations will require the dredge to suspend operation and to part the floating pipeline to allow vessels to traverse the location; this requires attendant tug service. As mentioned earlier, length of pipeline is a factor affecting the rate of dredging. When a length of pipeline is desired which is so great as to have a substantial stultifying effect on dredging rate, a booster pump can be introduced at an appropriate place in the line. Pipeline placement permits the maximum possible beneficial use of the dredged material.

VII-14. A hopper dredge may be operated without attendant plant. Material dredged is pumped into the vessel's hoppers for later discharge at the dumping ground. Because the dredge is not engaged in dredging while en route to and from the dumping ground, the cost of the operation increases with increase in running time between the cut and the dumping ground.

Disposal of Dredged Material

Open water disposal

VII-15. In open water disposal, the dredged material is deposited in unconfined disposal areas offshore. The effluent from a pipeline dredge can be discharged in any depth of water, but obviously hopper dredge and scow operations require adequate depths for dumping and maneuver. Open water areas are generally exposed and subject to currents and wave action. The duration, direction, and magnitude of these forces, and the characteristics of the material dumped determine the distribution pattern.

VII-16. Potential flow of the dredged material back into the channel must be recognized and fully considered in dumping outside the channel in waterways. This may happen even when the dumping ground is at sea, for it may be subject to forces which transport the spoil back to the channel. Studies of several cases strongly indicate that shoaling of the entrance channel has, in part, been caused by the return to the dredged channel of material dumped in deep water offshore.

VII-17. Open water disposal is sometimes accomplished by pipeline dredges discharging freely at distances of a mile or more from the dredged cut. The intention is to spread the material at so great a distance from the channel that its return is precluded. Hopper dredge loads and scow loads of dredged material are sometimes dumped in open water. When they are, the dumping grounds are usually located in naturally deep areas of the waterway. The forces that acted to create these deeps immediately remove the material dumped except coarse heavy particles of blasted rock, and it is likely that much of it returns to the areas requiring recurring dredging. In the past, a method of operating hopper dredges known as "agitation dredging" was extensively practiced; it is still occasionally utilized in some areas. Agitation dredging by hopper dredge was a process that, it was hoped, would result in successful open water disposal. The dredge pumps lift the material from the bottom, but it is then intentionally discharged overboard instead of being stored in the bins. The operation was practiced with the view that a major portion of the material would be transported by the currents to and deposited at locations beyond the channel, or that it would remain in suspension until swept out to sea. It is claimed that this method of open water discharge has been successful in some cases. In some cases, it may have been a useful temporary expedient. Perhaps it can be successful if judiciously done near enough the ocean during the early phases of the ebb current so that a reasonable proportion of the material discharged will be carried sufficiently far to sea before it sinks into bottom water strata to be beyond any preponderance of flood currents or ebb currents in these strata. Generally, however, the operation has been unsuccessful in that the material dumped is returned to the shoals that must be dredged to maintain the navigation channel.

VII-18. Another method of open water disposal has appeared recently. The plant utilizes unusually powerful pumps discharging through overboard discharge pipes suspended by booms extending (in one case) as far as 328 ft out from the side of the dredge. The basic principles are that the removal of very large quantities of material a fair distance from the channel at very low cost will be economical even though a substantial percentage returns to the channel shoals.

Upland disposal

VII-19. Upland disposal is accomplished by pipeline dredges only. The disposal areas usually require diking to retain the mixture of solids and water for sufficiently long periods of time for the solids to settle. The water flows out of the area through overflow weirs. The height of the retaining dikes, the location and construction of the weirs, the size of the settling basin, the distance between the outlet pipe and the weir, the discharge and settling rates, and the character of the material are the major factors in determining the effectiveness of upland disposal areas in retaining the fill. Studies of actual practice in numerous cases have shown that large quantities of maintenance dredging consisted of material which had escaped from upland disposal areas during former dredging operations and returned to shoal the channel. On numerous projects, high maintenance costs are the result of poor design and control of the upland disposal areas.

Rehandling methods

VII-20. When available upland disposal areas are too distant for the pipeline dredge to pump directly into the area, booster pumps are installed in the pipeline when this procedure is feasible. Where conditions are such that the employment of a pipeline dredge on the in situ material is not feasible, the dredging is sometimes accomplished by a hopper dredge and the material is dumped in a rehandling basin where it is pumped to the upland disposal by a pipeline dredge. Unless the rehandling basin is confined (as in the case of the Craney Island Disposal Basin at Hampton Roads, Virginia), considerable material is lost in the rehandling process.

Selection of Type of Operation

VII-21. Ordinarily the selection of the appropriate plant for use in the accomplishment of a specific dredging job will be readily apparent to an experienced person from the essential facts of the job; however, the occasional unusual job is the one which offers a challenge. The principal considerations upon which the selection is made are:

a. Exposure of the dredging site.

b. Volume and distribution of the material to be dredged.

c. The classes of material to be dredged.

d. Location of disposal area.

e. Distance to disposal area.

f. Time available for the work.

g. Vessel traffic.

h. Equipment that is or can be made available.

VII-22. Exposure is a matter of both geographical location and season of the year. The significant factor is the probability of swells in the area which would cause the dredge to pitch, with possible resultant damage to the plant or the parting of the pipeline. If critical swells are expected to prevail, the choice of equipment would be limited to a bucket dredge or a hopper dredge. If swells may occur for limited periods as the result of infrequent seasonal storms, and if a safe lee anchorage is available nearby, other types of dredges may be used, with the expectation that operation will have to be suspended on the occasion of intolerable seas and that the plant will have to be towed to safe haven to await abatement of the adverse condition.

VII-23. If the material to be removed comprises minor shoals lying in a scattered or extended pattern, removal by a hopper dredge would be the least costly because of the characteristic of a hopper dredge to load while "under way." If the material affords a dredging face of not less than about 5 ft and is disposed over a width of cut of about 150 ft or more, to permit advance by swinging, it would seem to be ideally suited for removal by a pipeline dredge, but could be removed by any type of dredge. The depth of water over the shoal is significant too if it is reduced to a point approaching the loaded draft of hopper dredges, the smallest of which (PACIFIC, 500 cu yd) has a loaded draft of 11 ft 3 in, and the largest (ESSAYONS, 8270 cu yd) a loaded draft of 31 ft. Depths less than the loaded draft prohibit the use of hopper dredges.

VII-24. A dredge with powered digging action, such as a dipper dredge or ladder dredge, would be well suited for digging packed sand, tough clay, or soft or broken rock without blasting. This material also could be handled by a pipeline dredge with an adequately powered cutterhead, or except for unbroken rock by a hopper dredge equipped with an appropriate drag head. Relatively hard rock would have to be blasted. It can then be removed by almost any type of dredging equipment.

VII-25. If the material is so fine or is so light that it does not settle out readily, it will present a problem with whatever type of dredge is used.

Normal operation of a hopper dredge would result in the material running back through the scuppers, normal pipeline dredging would result in the material running back over the disposal area weir, and dredging by dipper or bucket dredges would result in loss of material from the buckets or from the hoppers of the scows. In such a situation, provision for complete retention and removal of all material pumped should be considered.

VII-26. In cases requiring spoil to be placed in upland disposal areas, it must be pumped in. This requirement, then, indicates the selection of a pipeline dredge or, in special cases, resort to some combination technique such as the "sump rehandler" or the direct pump ashore operation from a hopper dredge described later. If upland disposal areas are so remote from the site of the dredging as to require too great a length of pipeline, even considering boosters in the line, then one of the options of open water dumping (already discussed) becomes necessary. Except where it is economical to transport the material to sea, a pipeline dredge could be used. If there is sufficient depth of water, a hopper dredge could be selected or under almost any conceivable condition, one of the dredges utilizing dump scows may be used. If the dumping ground is so remote as to require an impracticable length of pipeline, the pipeline dredge is eliminated from the consideration.

VII-27. The total yardage to be removed is a significant factor in the selection of a dredge because of the cost involved in mobilizing a dredging operation. As noted, a hopper dredge is a piece of plant that can operate independently, and therefore has a relatively low mobilization cost. If other considerations are not controlling, then the hopper dredge would be an economical choice for a small yardage job. If a great quantity of material is involved, the cost of mobilizing a pipeline dredge might be justified. If a great quantity of material is to be removed and the time of performance is limited, either by contract provisions or seasonal sea and weather conditions, multiple dredging units may be mobilized. These may be units of similar type or, if the work conditions are diverse, they may be units peculiarly suited to their respective types of digging. For example, the solution to a single problem might consist of (a) hopper dredge working in the outer channel because of the prevalent swells and dumping in deep water on the downdrift side of the channel, (b) pipeline dredge digging in the inner channel because of the considerable quantity of material that can be dredged at an economical cost and to realize an incidental benefit from land reclamation in an upland spoil area, and (c) bucket dredge working in the slips between the piers because of its ability to work in close quarters, dumping its material into scows for towing to open water dumping grounds, or preferably, to an enclosed basin for rehandling ashore by a pipeline dredge.

VII-28. Dredging operations must make full allowance for other vessel traffic. In the case of a pipeline dredge, if the pipeline crosses navigation routes it is necessary to discontinue digging, flush the line, break the line, and swing a portion of the floating line clear of the path of an approaching vessel. This is a costly delay, and if traffic is heavy, might be a deciding consideration in the choice of operation. When justified by the density of interrupting traffic and under otherwise favorable conditions, the portion of the pipeline crossing shipping lanes may be laid on the bed of the waterway and operated in this sunken condition. Similarly, a bucket dredge working on anchor lines for stability and maneuverability would constitute an obstruction to vessel traffic and must discontinue operations and slack off the anchor lines to permit approaching craft to pass over the lines.

VII-29. The foregoing considerations influence the selection of dredging equipment, but an additional practical consideration, which sometimes is of paramount influence, is the availability of the plant. The selected ideal plant often may be found to be committed elsewhere for the same period for which it is desired, or it may be located at a point which would entail unwarranted costs for mobilization. Except for the transfer of a hopper dredge, which can move as freely as any oceangoing vessel (and also can carry a considerable quantity of stores and spares), the towing of a dredge on the open seas can be done only after extensive readying for such an operation. Some of the newest dredges are built with a seagoing bow designed especially to facilitate ocean towing from job to job.

Recent Developments

VII-30. Disposal of the dredged material always has been recognized as one of the most important portions of the dredging operation, but much more attention has been given to the relation between recurring shoaling and disposal in the past decade than perhaps in all of the prior history of dredging. Oddly, the results of this thinking have gone off in two widely divergent directions and each appears to be meritorious. Both pertain especially to situations where the use of a pipeline dredge pumping directly ashore is impracticable.

VII-31. One of the methods evolved assures that all of the material dredged from the shoal in the channel by a hopper dredge is removed from the waterway and retained behind dikes. Initially, the thinking led to the construction of confined rehandling basins where a pipeline dredge could operate in water undisturbed by wave action or currents to remove material deposited by the hopper dredge. This plan had serious limitations, for it is not possible to build confined basins economically at any desired location, and it resulted in an inflexibility that was quite undesirable. There then evolved a plan whereby the hopper dredge would pump the contents of her hoppers into a floating sump equipped with powerful machinery to pump the material through a pipeline to an embanked area on shore (or in some cases, within the waterway where coves of no significance to the hydraulics of the waterway existed). The rate of discharge of the hopper dredge was carefully controlled so as to avoid exceeding the storage capacity of the sump, which came to be known as the "sump rehandler." The sump rehandler was moored on the land side of two strong mooring dolphins with her pipeline reaching shoreward to the disposal area, and the hopper dredge was moored at the channel side of the dolphins. A discharge pipe with a 90-deg turn at its end was lowered to position above a distribution box aboard the sump rehandler and then the pumps were started.

VII-32. Coincident with this development, exceptional care was taken to assure that the material pumped to the disposal area by the sump rehandler remained there to the maximum extent practicable. This involved watchfulness at the weir discharge to make certain that the solids content was not excessive, and patrol of the banks of the disposal area to make certain that they remained intact as the height of the fill increased. When it was found that the solids discharge at the weir was increasing to undesirable concentrations, the retention period was increased by raising the edge of the weir. Other measures were taken to reduce the "agitation" caused by the dredge during operation in the channel.

VII-33. It soon became evident that the basic principles of the efforts to retain all of the dredge spoil behind banks were sound, and that an improvement in the economics could be made by eliminating the sump rehandler from the system in favor of a hopper dredge that could competently pump the material ashore. The results of this innovation are very promising.

VII-34. In contrast to the thinking that has resulted in the plant and the operational procedures described previously is the idea which resulted in the so-called side-casting (or boom-discharging) plant. Reduced to its simplest dimensions, this method of dredging is premised on the philosophy that the most economic method to maintain channels under certain circumstances is to use a plant that can remove material from the channel much more rapidly than the natural processes bring material to the recurring shoal, and do so more cheaply than dredging involving conventional disposal methods. Proponents of this method do not assert that most of the material cast some distance away from the channel by means of the boom (as much as 328 ft in the latest item of a plant of this type placed on the job) is either carried away or becomes permanently settled at the point of discharge. They believe instead that the dredge can remove the natural shoaling material and that portion of the previously dredged material that returns to the shoal so rapidly and cheaply that the method is more economical than the conventional dredging and disposal methods. It is somewhat analogous to snow removal by the use of a powerful blower on a road traversing a windy vast prairie with a deep cover of dry loose snow. It would be foolish to shovel the snow into trucks and haul it away; the rate of supply is so large that the slight increment due to the availability of the snow deposited by the blower for a quick return to the road would not appreciably affect the rate at which the snow accumulates on the road.

VII-35. Although privately owned existing items of a plant embodying the principles described above are very powerful, it is not axiomatic that all plants to be operated in this manner need be large and powerful. Instead, the ideal plant for each case would be competent to remove the material at a rate greater than that at which it accumulates naturally, plus an additional capacity to remove the increment to the natural shoaling rate due to the return of previously dredged material. The Corps of Engineers contemplates the construction of one fairly large dredge (the McFARLAND) and two small dredges having this capability, in addition to the existing dredge HAINS.

VII-36. It is apparent that a side-casting dredge is not a proper tool for the maintenance of the channels in estuaries such as the Savannah, the Delaware, and the Hudson. Some of the material removed from the channel would remain in place even though most of it returned to the shoal to be dredged again and some portion of the remainder was carried some distance away. Over a long period, the material remaining would accumulate and form a shoal that would seriously obstruct access to landing facilities alongshore. Further, some of the material deposited just outside of the channel would probably contribute to the shoaling of the slips and marginal wharf frontages. Finally, the mechanics of these streams are such that there is no possibility that some of the material would ultimately reach the sea; the portion that leaves the immediate area of the dredging may migrate downstream some distance, but ultimately it will be trapped at the point in the estuary where the upstream predominance of the lower strata of flow is first encountered. Nevertheless, the method seems to have a place in the maintenance of channels where the accumulation of a bar just beyond the channel limits is unobjectionable, or where a crosscurrent generally in one direction exists, as for example at an outer bar beyond the mouth of a great river or at the entrance to a lagoon.

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CHAPTER VIII
DISPERSION AND FLUSHING OF POLLUTANTS

by

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Introduction

VIII-1. This chapter treats the processes of movement and dispersion of a waterborne pollutant within and through a tidal waterway. The treatment is limited for the most part to foreign material which occurs in the waterway in dissolved form, or which is completely miscible with the receiving waters and does not separate from them under the action of gravity. Consideration is given to the case of nearly instantaneous discharge to the waterway of a given amount of pollutant, and to the case of a continuous discharge of polluted effluent, as from an outfall pipeline, into the waterway. The subsequent movement and spread of the contaminated volume within and through the waterway for these two types of sources is discussed.

General Considerations

VIII-2. Consider that a given amount of material M contained in an aqueous solution of volume V_o is introduced over a short time period into a tidal waterway. The concentration of the effluent prior to entering the waterway is then M/V_o, and theoretically this is the initial concentration of the pollutant in the immediate vicinity of the point of introduction. Practically, however, this concentration does not occur over any finite time interval or space interval, since the process of introduction of the pollutant into the receiving waters will itself result in some initial mixing between the pollutant and the receiving waters. The initially contaminated volume is then somewhat larger than V_o, and the initial maximum concentration within the waterway somewhat less than M/V_o.

VIII-3. The effectiveness of this initial mechanical dilution will depend upon the density difference between the undiluted pollutant volume and the receiving waters, and upon the manner of introduction of the pollutant volume into the waterway. If the density of the pollutant volume differs materially from that of the receiving waters, the initial mechanical dilution, as well as later dispersion by turbulent diffusion, will be inhibited. If the effluent is introduced into the waterway as a subsurface jet of small cross section, greater initial mechanical dilution will occur than in the case of introduction as a gentle broad flow at the surface or near the bottom. A pollutant volume which is less dense than the receiving waters will spread out on the surface, and one which is more dense will spread out over the bottom. If a pollutant volume less dense than the receiving waters is introduced near the bottom of the waterway, mechanical dilution will be augmented by the convective rise of the volume through the receiving waters. Likewise, a pollutant volume denser than the receiving waters, when introduced at the surface, will sink through the water column entraining dilution waters on the way to the bottom. Thus, the effect of a density difference between the undiluted pollutant volume and the receiving waters in restricting initial mechanical dilution may be at least partially offset by proper location of the depth of the discharge point in order to take advantage of the convective entrainment of dilution waters.

VIII-4. In most cases the influence of the initial density difference between the undiluted pollutant volume and the receiving waters is limited to a relatively small area near the point of discharge. It is common practice in industry to mix high-concentration waste streams with relatively large volumes of cooling water or of process water having low contaminant concentration prior to discharge. As a result, the initial polluted volume has contaminant concentrations which seldom exceed a few thousand parts per million, and more commonly are on the order of a few hundred parts per million. Municipal sewage discharge has a dissolved solids concentration which is on the order of a hundred parts per million. Except in the uppermost reaches of the estuary, where the salinity may be relatively small, waste volumes introduced into estuaries are usually less dense than the receiving waters. In some cases industry will mix high-concentration waste streams with cooling water which has been drawn directly from the estuary. The resulting effluent then would have a slightly higher concentration of dissolved solids than the receiving waters, and if at the same temperature as the receiving waters, would have a slightly higher density. However, in these cases the effluent is generally at a sufficiently higher temperature than the receiving waters so that the effluent is less dense than the estuarine waters. The initial mechanical dilution in the case of an introduction directly into the surface layers, or the combined mechanical plus convective dilution if such polluted volumes are introduced into the bottom layers, is usually sufficient so that, except within a few hundred yards of the discharge point, the difference in density between the contaminated volume and the surface layers is not sufficient to affect the subsequent processes of movement and diffusion. In subsequent paragraphs the movement and dispersion of the contaminated volume resulting from initial mechanical dilution and any convective entrainment of dilution waters are discussed. Any residual density difference between this diluted contaminated volume and the immediate surrounding receiving waters is, for purposes of this subsequent discussion, considered negligible.

VIII-5. After initial mechanical dilution, the contaminated mixture of pollutant and receiving waters will be transported by the currents in the waterway, and will spread both vertically and horizontally by turbulent diffusion. Tidal currents will carry the contaminated volume back and forth in the waterway over the tidal excursion, and the contaminated volume will participate in the net nontidal motion. The oscillatory motion of the tide is important in supplying the major portion of the turbulent energy which leads to diffusion. The oscillatory motion of the pollutant with the tidal currents past irregularities in the shoreline, such as embayments and protruding points of land, leads to a longitudinal dispersion of the pollutant in a manner to be described in a later paragraph.

VIII-6. The processes of turbulent diffusion lead to a reduction in the concentration of the contaminated volume with time, and lead to a net transport of pollutant from regions of high concentration toward regions of low concentration. A presentation of the instantaneous spatial distribution of pollutant would reveal a rather irregular picture, with discrete masses of high concentration surrounded by lows. However, the smoothed horizontal distribution would be approximately bell-shaped, with a central area of relatively high concentrations and a regular decrease in concentration with distance from the center. With the passage of time the spatial distribution becomes less sharply peaked and spreads over an ever increasing area.

VIII-7. Vertical diffusion is most intense in layers having vertical homogeneity, i.e. where density remains constant over a range in depth. Vertical stability, which results from a rapid increase in density with depth, greatly inhibits vertical diffusion. In many estuaries the surface layers are of lower salinity, and hence less dense, than the deeper layers. In such cases, intermediate depths exhibit a rapid increase in salinity and density which limits the vertical turbulent diffusion. A pollutant introduced near the surface is rapidly mixed vertically within the surface layers, but is only slowly transferred through the region of vertical stability to the deeper layers. Likewise, material introduced into the deeper layers will be rapidly mixed vertically within the layers of weak vertical stability, but will only slowly diffuse into the surface layers through the intermediate depth region of relatively large density gradient.

VIII-8. When a pollutant is introduced as a continuous effluent, as from an outfall pipeline, the contaminated volume spreads down current in a plume, much like a smoke plume from a smokestack in the atmosphere. The oscillatory motion of the tide in a tidal waterway folds this plume back and forth upon itself, so that the evident features of a plume are masked except very close to the outfall. One convenient way of envisioning the spatial distribution of concentration of the pollutant from a continuous source is to consider the material discharged within each successive infinitesimal time interval as an instantaneous source which will move with the flow and disperse vertically and horizontally as a result of turbulent diffusion. The spatial concentration distribution at a given time is then the sum of the distributions of all the individual spreading instantaneous sources released prior to the given time.

The Effect of Oscillatory Tidal Motion on the Longitudinal Spread of Pollutant

VIII-9. Tidal oscillations past irregularities in the shoreline are an important mechanism in the longitudinal dispersion of an introduced pollutant. This is made most evident by considering a contaminated volume, produced by an instantaneous release of pollutant, as it is carried up and down the waterway by the tidal currents. Frequently, eddies associated with slight embayments or with points of land which project into the waterway will temporarily trap water containing a high concentration of pollutant as the contaminated volume moves past the shore features on one or the other phase of the tide. The main bulk of the contaminated volume is carried on past the shore feature by the tidal current, while the pollutant trapped by the shore feature slowly spreads out into the main stream, leading to an effective dispersal of the pollutant behind the bulk of the contaminated volume. When the tide reverses, the process is repeated, with a resulting dispersion on the opposite side of the contaminated volume. This mechanism is shown schematically in fig. VIII-1.

The Effect of the Net Nontidal Circulation on the Dispersion of an Introduced Pollutant

VIII-10. A discussion of the effect of the net circulation pattern on the dispersal of a pollutant introduced into a tidal waterway requires first some description of the nontidal motion in such waterways. The majority of the harbors along the coast of the United States are located in the lower reaches of drowned river valleys. Such tidal waterways are called coastal plain estuaries, and the discussion of the net nontidal circulation patterns will begin with these water bodies. First, however, it is desirable to clearly define what is meant by the term "estuary."

VIII-11. While there is not universal agreement as to the definition of an estuary, the following statement, which has received considerable support in recent scientific literature, is suitable for use in this chapter:

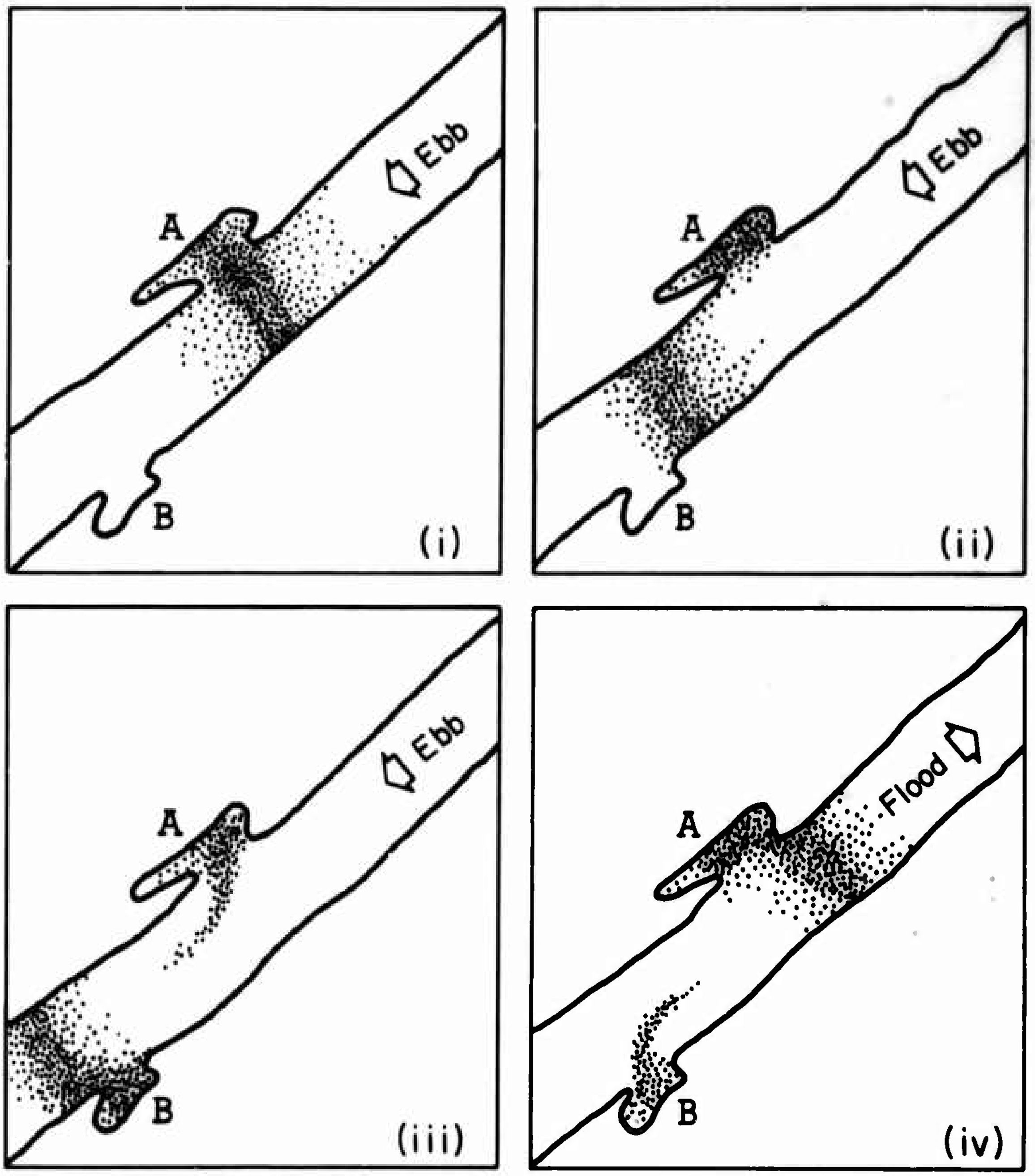

Fig. VIII-1. Schematic representation of entrapment by shore features. Concentration is indicated by intensity of shading. As peak concentration of pollution moves downstream on the ebb tide, a portion spreads into shore indenture A, in diagram (i). This portion is entrapped by the indenture as the main volume moves on downstream, as shown in diagram (ii). In diagram (iii), the entrapped pollutant feeds out into the main channel, thus contributing to the longitudinal spread of pollutant, and reducing the concentration. A portion of the main volume is shown spreading into a second shore indenture B. In diagram (iv), the phenomenon of delayed dispersal following temporary entrapment is repeated for indenture B on the flooding tide

> An estuary is a semienclosed coastal body of water, having free connection with open sea, and within which sea water is measurably diluted by freshwater drainage.

VIII-12. A coastal plain estuary is generally an elongated indenture in the coastline formed by the drowning, due to a relative rise in sea level, of the lower reaches of a river valley. The seaward extent of the estuary is usually well defined by prominent headlands or capes at the open coastline. Under the preceding definition, the landward extent of the estuary is marked by the upstream limit of measurable quantities of sea-derived salt. This upper limit of the estuary does not, however, coincide with the landward extent of tidal action. Normally, the predominant ebb and flood of the tidal currents occur for some distance above the limit of intrusion of any sea water, and the tidal rise and fall of the water surface may extend some distance above the upstream limit of flood current reversal. The segment of the freshwater river above the upper limit of sea-water intrusion, but within which tidal influence is still felt, is called the tidal section of the river in order to distinguish this region from the estuary proper.

VIII-13. Most coastal plain tidal estuaries can be grouped into four classes according to the physical structure of the water and to the net motion. In the following paragraphs the net circulation pattern and its effect on the distribution of an introduced pollutant in these four types of coastal plain estuaries will be discussed, after which a brief treatment of some tidal waterways which do not fit into this classification will be given.

VIII-14. In a salt-wedge estuary the river inflow is large compared to the tidal flow. Sea water enters as a wedge along the bottom, with the fresh water flowing seaward on top of the wedge. If no frictional drag existed between the denser saltwater wedge and the lighter river water, the wedge would extend upstream to the point where the river bottom intersected mean sea level. The river inflow would move seaward over the stationary wedge as a thin freshwater layer. In the real fluid being treated, some frictional drag always exists. The extent of intrusion of the wedge up the estuary depends upon the magnitude of this frictional drag, which in turn depends upon the relative velocities in the upper, seaward-flowing layer and in the wedge. Thus, the volume rate of flow of the river water which moves seaward on top of the saltwater wedge controls the position of the wedge. Under conditions of high river flow, the wedge extends only a short distance into the estuary, while for low river flows the wedge extends many miles upstream. The Mississippi River is an example of this salt-wedge estuary. When the flow in the Mississippi is low, the undiluted salt wedge extends upstream for over 100 miles. Under conditions of high river flow, the salt wedge extends only a mile or so above the mouth of the river.

VIII-15. Comparatively little mixing occurs at the interface between the seaward-flowing upper layer and the salt wedge; hence, the salinity throughout the wedge is nearly that of full sea water. At the upper boundary of the wedge, unstable interfacial waves form and break into the upper layers, producing a slowly increasing salt content in these layers as they move seaward. Even so, a very sharp vertical salinity gradient exists between the upper layer of relatively low salinity and the salt wedge. For a given river flow, the horizontal position of the wedge and its vertical dimension remain stationary (except for a relatively minor oscillatory movement upstream and downstream in consonance with the phases of the tide). The loss of salt water from the wedge to the upper layer must be compensated for by flow of water into the wedge from the sea. Since this loss of salt water to the upper layer takes place all along the upper boundary of the wedge, there must be, in order to maintain the position and shape of the wedge, a flow directed toward the upstream tip of the wedge at all positions within the wedge. Thus, the circulation pattern in this salt-wedge estuary involves a seaward-flowing upper layer riding over the landward directed flow in the salt wedge. The compensating flow in the wedge is generally small compared to the seaward flow of the low-salinity surface layers.

VIII-16. A pollutant volume which, after initial mechanical and convective dilution, occurs in the deep layers of a salt-wedge estuary, will be dispersed vertically and horizontally by turbulent diffusion within the wedge. Since tidal action is relatively weak in salt-wedge estuaries, horizontal diffusion associated with the oscillatory tidal currents will be relatively slow. The waste is carried by the slow upstream-directed flow to the very tip of the wedge. The process of upward exchange of water from the wedge slowly transfers contaminated water to the seaward-flowing upper layer. Once in the upper layer, the waste is carried seaward with the surface-water layers and transported out of the estuary. The time required to flush the wastes from the estuary in this case is relatively long, since the rate of transfer of the waste from the salt wedge to the surface layers is slow.

VIII-17. A pollutant volume which, after initial mechanical and convective dilution, occurs in the surface layers of a salt-wedge estuary is rapidly flushed from the estuary with the seaward-flowing surface waters. None of the contaminated water enters the salt wedge, since exchange across the upper interface of the wedge is essentially one way, directed from the wedge to the fresher surface layers.

VIII-18. In a partially mixed estuary, tidal movements are large compared to the river inflow, vertical mixing is sufficiently strong to destroy the sharp boundaries separating the salt wedge from the upper layer, and the wedge ceases to exist as a readily identifiable feature. There still exists a transition layer of relatively rapid increase in salinity with depth called the halocline, which separates the lower salinity surface layers from the higher salinity deeper water. The salinity in both the surface layers and the bottom layers decreases steadily from the mouth to the head of the estuary. Most of the estuaries along the eastern coast of the United States fall into this class of partially mixed estuaries.

VIII-19. The oscillatory tidal currents produce the most obvious motion in these partially mixed estuaries. Superimposed on the tidal currents there is a net nontidal circulation pattern with a net seaward flow in the surface layers and a net flow directed from the mouth toward the head of the estuary in the deeper layers. This net two-layered circulation pattern does not extend all the way up the estuary to the limit of sea-salt intrusion; the low-salinity reaches near the head of the estuary frequently exhibit characteristics of the vertically homogeneous estuaries described in later paragraphs. The discussion of partially mixed estuaries will be restricted to that segment of the estuary in which the net two-layered circulation pattern exists. In this segment there is also a small net vertical motion directed from the deeper layers to the surface layers. The volume of water flowing toward the head of the estuary per unit time decreases as one proceeds from the mouth to the head of the estuary since water is being transferred through vertical motion from these deeper layers to the surface layers. Hence, the volume rate of seaward flow in the surface layers increases as one proceeds from the head toward the mouth of the estuary.

VIII-20. Meteorological phenomena can lead to a temporary disruption of the net circulation pattern as described above. Thus, a strong, prolonged wind directly up the estuary may reverse the seaward-directed flow at the surface, producing a three-layered flow pattern with near-surface and bottom layers flowing toward the head of the estuary and the middepth layers flowing seaward. Such a wind may also produce a piling up of water within the estuary above the normal mean tide levels. When this landward-directed wind weakens or changes direction, the stored water in the estuary will flow out, producing a net nontidal flow directed seaward at all depths. However, averages of current velocities taken over long enough periods of time so that both the tidal and meteorological effects are averaged out reveal the characteristic two-layered pattern described here.

VIII-21. As noted in previous paragraphs, the volume rates of flow associated with the net nontidal circulation pattern increase as one proceeds toward the mouth of the estuary. In the lower reaches of the estuary, both the net seaward-directed flow in the upper layers and the net opposite-directed flow in the lower layers will frequently greatly exceed the freshwater inflow. The difference between these flows, i.e. the net flow through the total cross section, must of course just equal the freshwater inflow to the estuary above the section. This feature of partially mixed estuaries is very important from the standpoint of the flushing of pollutants introduced into the lower reaches of the waterway. The rate of addition of "new" water available for dilution of a pollutant introduced into a given segment in the lower reaches of the estuary frequently equals ten to twenty times the rate of freshwater inflow from the river.

VIII-22. A pollutant initially introduced into the bottom layers in a partially mixed coastal plain estuary, in addition to participating in the oscillatory movement of the tidal currents, is carried in the net motion toward the head of the estuary. At the same time, turbulent mixing leads to horizontal dispersion in both the longitudinal and lateral directions, and to vertical dispersion into the surface layers. The pollutant which becomes mixed with the surface layers is carried in the net flow toward the mouth. Seaward from the point of introduction, the pollutant being carried toward the ocean in the surface layers is partially mixed downward into the deeper layers and reintroduced into layers moving toward the head of the estuary.

VIII-23. A pollutant introduced into the surface layers is initially carried in the net flow toward the mouth of the estuary. Turbulent mixing leads to both horizontal and vertical dispersion, and the wastes are thus also added to the deeper layers having a net flow directed toward the head of the estuary.

VIII-24. In the region of the estuary headward from the point of introduction, the concentrations of the pollutant will always be greater in the deeper layers than in the surface layers while seaward from the point of introduction the converse will be true. These conditions prevail regardless of whether the wastes are initially introduced into the surface layers or into the deeper layers.

VIII-25. The pollutant is ultimately flushed from the estuary in the seaward-directed flow of the surface layers.

VIII-26. In estuaries in which the tidal currents are very large compared to the motion required to transport the inflowing fresh water to the sea, enhanced vertical mixing tends to sufficiently weaken the vertical variation in salinity so that the density-induced, two-layered flow pattern, characteristic of partially mixed estuaries, is replaced by a net nontidal movement toward the sea at all depths. Such estuaries are called vertically homogeneous estuaries, though in fact in many such waterways there remains a small increase in salinity with depth. The predominant feature of the salinity distribution is, however, the longitudinal decrease from the mouth to the head of the estuary. Also, in relatively wide estuaries in the northern hemisphere, the salinity on the right side (looking toward the mouth) will be lower than the salinity on the left side, as a result of the effects of the earth's rotation. For the southern hemisphere the converse is true.

VIII-27. The circulation pattern in a vertically homogeneous estuary shows no reversal in water movement with depth. In relatively wide estuaries, net seaward flow occurs along the right side (looking toward the mouth of the estuary), and a net motion directed toward the head of the estuary occurs on the left side. A laterally directed flow carries water from the left side of the estuary to the right, and large-scale horizontal mixing occurs between the counterflows on the two sides of the estuary.

VIII-28. A pollutant exhibiting no marked density difference with the receiving waters will rapidly show a uniform depth distribution when introduced into a vertically homogeneous estuary. If the point of introduction is on the right side of the estuary (looking seaward), the contaminated volume will be carried toward the mouth with the net flow. At the same time, lateral turbulent diffusion will lead to a transport of some of the pollutant into the waters on the left side of the estuary. Here the pollutant will be carried toward the head of the waterway in the net flow pattern.

VIII-29. If the point of introduction is on the left side of the estuary, the contaminated volume is initially carried toward the head of the waterway in the net flow pattern. The transverse flow plus lateral diffusion provides for a transport of pollutant to the right side of the estuary, where the predominant flow is seaward.

VIII-30. The pollutant is ultimately flushed from the estuary in the predominant seaward flow on the right side of the estuary.

VIII-31. Sectionally homogeneous estuaries are comparatively narrow, vertically homogeneous waterways where tidal mixing is sufficient to destroy the lateral salinity gradient. The predominant variation in salinity is the decrease from the mouth to the head of the estuary. The net circulation pattern is quite simple, being a slow seaward movement at all depths. The estuary of the Severn in England has been cited in the literature as a sectionally homogeneous estuary. Also, some of the small tributary estuaries to the Chesapeake Bay exhibit the characteristics of a sectionally homogeneous estuary during some parts of the year. The upper reaches of many partially mixed estuaries, where the salinity becomes quite small and cross section quite narrow, also show the characteristics of a sectionally homogeneous estuary.

VIII-32. In such estuaries an introduced pollutant is rapidly mixed vertically and horizontally. The contaminated volume is carried back and forth with the oscillatory motion of the tidal currents, is dispersed up and down the estuary by turbulent diffusion, and is slowly carried seaward with the net flow.

VIII-33. Many drowned river valleys have an appreciable stretch above the limit of sea-salt intrusion which is still subjected to tidal action. In such tidal waterways the freshwater reach immediately upstream from the upper limit of sea-salt intrusion exhibits both ebb and flood flow. In some cases, as for example the Delaware, the James, and the Potomac, the cross-sectional areas in this reach are sufficiently large so that the net downstream flow required to transport the freshwater inflow seaward is small compared to the peak of flood and ebb flows. As one proceeds upstream, the magnitude and duration of the flood flows decrease relative to the ebb flows until a point is reached at which slack water occurs once each tidal cycle; at all other times in the cycle, the flow is directed downstream, or in the ebb direction. Upstream from this position there is no reversal of flow; the downstream flow simply shows a time variation of tidal period. This periodic fluctuation in current speed decreases as one proceeds upstream. At a point near where the river bottom rises above sea level, both the tidal rise and fall of the water surface and the periodic tidal fluctuation in the downstream current speed disappear.

VIII-34. Some vertical stratification may occur in the tidal section of the river as a result of surface layer heating in spring and early summer. This effect is seldom sufficiently strong to influence the movement and dispersion of an introduced pollutant. This section of the tidal waterway thus exhibits the same characteristics with respect to flow pattern and turbulent diffusion as described in the previous paragraphs for a sectionally homogeneous estuary.

VIII-35. Baltimore Harbor is an example of a tidal waterway having a net nontidal circulation pattern quite different from those described for the four characteristic estuarine types discussed previously. This waterway is a tributary embayment to the upper Chesapeake Bay. The waters of the Bay adjacent to the mouth of the Harbor exhibit the characteristics of a partially mixed estuary. A moderately strong vertical salinity gradient is maintained by the mixing of the inflowing Susquehanna River water with sea water moving up the Bay. The characteristic two-layered flow pattern exists in this section of the Bay, with a surface layer having a net nontidal movement toward the sea and a deeper layer having a net nontidal movement toward the head of the Bay. The vertical difference in salinity between the surface and a depth of 40 ft averages about 8 parts per thousand and varies seasonally from about 2 to about 14 parts per thousand.

VIII-36. The tributary embayment containing Baltimore Harbor is sometimes called the Patapsco River estuary. However, the freshwater inflow from the Patapsco River plus other small streams entering the Harbor is very small and has little effect on the salinity distribution and circulation pattern of this waterway. The water that fills the Harbor originates primarily from the adjacent Bay. Within the Harbor the mechanism which maintains the vertical salinity gradient and drives the two-layered flow pattern characteristic of the Bay does not exist. Vertical mixing weakens the vertical variation in salinity. There is virtually no longitudinal variation along the length of the embayment in the vertical averaged salinity. However, because vertical mixing has reduced the vertical variation in salinity within the Harbor, as compared to the adjacent Bay waters, the surface layers in the Harbor are more saline than in the Bay, while the bottom layers in the Harbor are less saline than the adjacent Bay waters at the same depth. Fig. VIII-2 shows the typical longitudinal salinity distribution in a vertical section taken along the navigation channel between the mouth of the tributary embayment and the head some eight miles up the Harbor.

VIII-37. Because of density effects, the lighter, lower salinity surface waters of the adjacent Bay tend to flow into the Harbor in the surface layers; the bottom waters of the Bay, being of higher salinity and hence more dense than the bottom waters of the Harbor, also tend to flow into the Harbor. These combined inflows are returned to the Bay in an outflow of the middepth layers. This three-layered circulation pattern is shown schematically in fig. VIII-2.

VIII-38. Such a net circulation pattern can be a very effective mechanism for dispersal and flushing of pollutants introduced into tributary embayments such as Baltimore Harbor. In that waterway the local freshwater inflow would require about 200 days to replace the volume of the Harbor. However, the density-driven, three-layered flow pattern described previously provides for the renewal of about ten percent of the Harbor volume each day.

VIII-39. Some tributary embayments adjacent to strong partially mixed estuaries do not have channel depths sufficiently great to develop the three-layered flow pattern described above. The Magothy estuary, which is tributary to the Chesapeake Bay just to the south of Baltimore Harbor, is an example of this type of embayment. As in the case of Baltimore Harbor, the local freshwater inflow into the Magothy is relatively small, and the waters filling this embayment originate primarily from the adjacent Bay. However, the Magothy is only about 18 ft deep, and hence has horizontal contact only with the Bay waters above the halocline. In this type of tributary embayment, the net circulation pattern is driven by the horizontal salinity gradients set up by time changes in the salinity of the surface layers of the adjacent estuary. The waters of the Chesapeake Bay adjacent to the mouth of the Magothy are subjected to seasonal variations in salinity resulting from the large seasonal changes in river inflow to the head of the Bay. The salinity within the Magothy shows a lag in seasonal change with respect to the adjacent surface layers of the Bay. Thus, during the winter and spring period when the salinity of the Bay waters is decreasing with time, the salinities within the Magothy are higher than those of the adjacent Bay waters. On the other hand, during summer and fall, when the salinity in the Bay is

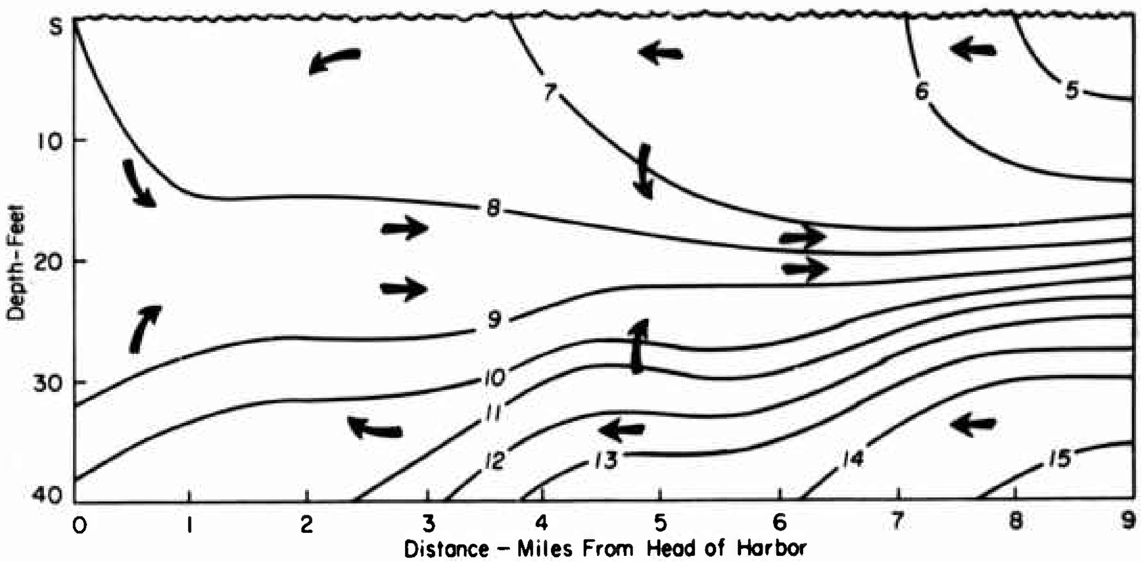

Fig. VIII-2. Typical longitudinal section of the salinity distribution (in parts per thousand) in Baltimore Harbor. The Chesapeake Bay at the mouth of the Harbor is at the right end of the figure. The arrows show the net flow pattern increasing, the salinities in the Magothy are lower than in the adjacent Bay. During the winter and spring period, the lower-salinity Bay waters flow into the Magothy on the surface, and there is a return flow from the Magothy to the Bay along the bottom. During the summer and fall season, the reverse is true, with the lower-salinity surface layers of the Magothy flowing out into the Bay at the surface, and a return inflow to the Magothy takes place along the bottom of that waterway.

VIII-40. At some time in early summer and again in late fall, the salinities in the tributary embayment and the adjacent Bay waters become equal, and no net nontidal circulation pattern exists for exchange of water between the Magothy and the Chesapeake Bay. At these periods the flushing rates for the tributary embayment are very low, depending only on tidal exchange and wind-driven motion.

Theoretical Considerations
General Case of Three Spatial Dimensions

VIII-41. Two processes control the local time rate of change of concentration of a conservative, waterborne pollutant. These processes are <u>advection</u> and <u>diffusion.</u> The advective term involves the product of the velocity times the local concentration gradient. If the instantaneous values of velocity and concentration could be measured, and of even more importance, if the time variation in the instantaneous velocity field could be reasonably depicted, then the diffusion term which would enter the considerations would be the molecular diffusion. The motion in a tidal waterway is, however, turbulent in character. It is not possible to treat the instantaneous field of motion; rather, time mean velocities must be used. The time period used in obtaining the time mean may be short, of the order of approximately 10 min, or may extend over a tidal cycle, depending on the purpose of the evaluation.

VIII-42. The processes of turbulent diffusion in a tidal waterway are manyfold larger than the processes of molecular diffusion, and the latter term is normally neglected in evaluating the dispersion of a pollutant. The basic differential equation expressing the time rate of change of concentration is developed from continuity concepts. This development has been adequately treated in the literature,[1,2,3*] and only the resulting differential equation, together with its application to dispersion of a contaminant in a tidal waterway, will be presented here.

VIII-43. The frame of reference here will be a right-handed rectangular

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* Raised numerals refer to similarly numbered items in Literature Cited at the end of this chapter.

coordinate system oriented so that the x_1 axis is directed longitudinally along the central axis of the tidal waterway, the x_2 axis is directed laterally across the waterway, and the x_3 axis is directed vertically downward. The x_1, x_2 plane is taken at mean tide level, and the origin is placed at the landward end of the waterway. Further,

ζ = concentration of pollutant at the point x_1, x_2, x_3 and the time t

v_1, v_2, v_3 = the x_1, x_2, and x_3 components of the velocity at the point x_1, x_2, x_3 and the time t

K_1, K_2, K_3 = the x_1, x_2, and x_3 components of the eddy diffusivity at the point x_1, x_2, x_3 and the time t

VIII-44. The local time rate of change of concentration of the pollutant is expressed by:

$$\frac{\partial \zeta}{\partial t} = -v_1 \frac{\partial \zeta}{\partial x_1} - v_2 \frac{\partial \zeta}{\partial x_2} - v_3 \frac{\partial \zeta}{\partial x_3} + \frac{\partial}{\partial x_1}\left(K_1 \frac{\partial \zeta}{\partial x_1}\right) + \frac{\partial}{\partial x_2}\left(K_2 \frac{\partial \zeta}{\partial x_2}\right) + \frac{\partial}{\partial x_3}\left(K_3 \frac{\partial \zeta}{\partial x_3}\right) \qquad \text{(VIII-1)}$$

This general three-dimensional equation states that the local time rate of change of concentration of a conservative constituent is equal to the effects of advection (first three terms on the right side of the equation) plus the effects of turbulent diffusion (last three terms on the right side of the equation). No solution of this general form is available; in practice it is necessary to assume that one or more of the spatial dimensions may be neglected, and to treat an equation of reduced complexity. No general theory exists for the evaluation of the diffusivity coefficients, which may be functions of both time and space. Both the concentration and the velocities in this equation are mean values over designated time and space intervals, rather than instantaneous point values. The magnitudes of the diffusivities are evidently related to this averaging process.

VIII-45. The equation of mass continuity is an important auxiliary relation when equation VIII-1 and expressions derived from equation VIII-1 are employed. Water may be considered as an incompressible fluid, for which the equation of continuity takes the form:

$$\frac{\partial v_1}{\partial x_1} + \frac{\partial v_2}{\partial x_2} + \frac{\partial v_3}{\partial x_3} = 0 \qquad \text{(VIII-2)}$$

From this relation it is seen that equation VIII-1 may also be written:

$$\frac{\partial \zeta}{\partial t} = - \frac{\partial v_1 \zeta}{\partial x_1} - \frac{\partial v_2 \zeta}{\partial x_2} - \frac{\partial v_3 \zeta}{\partial x_3}$$

$$+ \frac{\partial}{\partial x_1}\left(K_1 \frac{\partial \zeta}{\partial x_1}\right) + \frac{\partial}{\partial x_2}\left(K_2 \frac{\partial \zeta}{\partial x_2}\right) + \frac{\partial}{\partial x_3}\left(K_3 \frac{\partial \zeta}{\partial x_3}\right) \qquad \text{(VIII-3)}$$

By analogy to the equation of mass continuity, equations VIII-1 and VIII-3 are here called equations of pollutant continuity.

Two-Dimensional Tidal Waterway

VIII-46. Some tidal waterways are very nearly vertically homogeneous, and the spatial variations in velocity and salt concentrations are limited to the two horizontal coordinates. Other systems exhibit little lateral variation, and significant gradients in the fields of velocity and density exist only along the longitudinal and vertical axes. In both these situations the pertinent equations may be reduced to two dimensions.

VIII-47. In the case of the vertically homogeneous tidal waterway, having a local depth designated by $h = h(x_1, x_2, t)$, the equation of mass continuity reduces to:

$$\frac{\partial}{\partial x_1}(hv_1) + \frac{\partial}{\partial x_2}(hv_2) + \frac{\partial h}{\partial t} = 0 \qquad \text{(VIII-4)}$$

and the equation of pollutant continuity reduces to:

$$\frac{\partial (h\zeta)}{\partial t} = - \frac{\partial}{\partial x_1}(h v_1 \zeta) - \frac{\partial}{\partial x_2}(h v_2 \zeta) + \frac{\partial}{\partial x_1}\left(K_1 h \frac{\partial \zeta}{\partial x_1}\right) + \frac{\partial}{\partial x_2}\left(K_2 h \frac{\partial \zeta}{\partial x_2}\right) \qquad \text{(VIII-5)}$$

which, by virtue of equation VIII-4, may also be written:

$$h \frac{\partial \zeta}{\partial t} = - hv_1 \frac{\partial \zeta}{\partial x_1} - hv_2 \frac{\partial \zeta}{\partial x_2} + \frac{\partial}{\partial x_1}\left(K_1 h \frac{\partial \zeta}{\partial x_1}\right) + \frac{\partial}{\partial x_2}\left(K_2 h \frac{\partial \zeta}{\partial x_2}\right) \qquad \text{(VIII-6)}$$

VIII-48. In the case of the laterally homogeneous tidal waterway, having a local width designated by $w = w(x_1, x_3)$, the equation of mass continuity becomes:

$$\frac{\partial}{\partial x_1}(wv_1) + \frac{\partial}{\partial x_3}(wv_3) = 0 \qquad \text{(VIII-7)}$$

and the equation of pollutant continuity reduces to:

$$\frac{\partial(w\zeta)}{\partial t} = - \frac{\partial(wv_1\zeta)}{\partial x_1} - \frac{\partial(wv_3\zeta)}{\partial x_3} \qquad \text{(VIII-8)}$$

which, by virtue of equation VIII-7, may also be written:

$$w\frac{\partial\zeta}{\partial t} = - wv_1 \frac{\partial\zeta}{\partial x_1} - wv_3 \frac{\partial\zeta}{\partial x_3} + \frac{\partial}{\partial x_1}\left(K_1 w \frac{\partial\zeta}{\partial x_1}\right) + \frac{\partial}{\partial x_3}\left(K_3 w \frac{\partial\zeta}{\partial x_3}\right) \qquad \text{(VIII-9)}$$

VIII-49. Even these reduced forms of the continuity equations remain too complex to treat analytically. There is some promise in the use of high-speed electronic computers to solve these equations numerically for specific cases. A necessary requirement is the determination of the eddy diffusivities K_1, K_2, and K_3. Since these equations express the continuity of salt, as well as any other conservative quantity, the procedure generally employed is to utilize the observed salinity distributions, together with the velocity distributions, to determine the diffusivities. This procedure is complicated by the observation that the horizontal diffusivities are apparently a function of the scale of the diffusing phenomena. Thus, coefficients of eddy diffusion applicable to the dispersion of a contaminated volume which is small compared to the dimensions of the waterway would be much less in magnitude than the coefficients characterizing the distribution of salt in the tidal waterway. This feature will be discussed more fully in later paragraphs.

VIII-50. Pritchard[2] utilized equations VIII-7 and VIII-9 in a study of the James River, a tributary estuary to the Chesapeake Bay. The area of study extended from the mouth of the estuary, where the salinities during the period of study were about 20 parts per thousand, to a position some 24 miles up the estuary, where the salinities were about 5 parts per thousand. In this major stretch of the estuary there exists, superimposed on the oscillatory tidal motion, a net circulation pattern having a net flow directed toward the mouth of the estuary in approximately the upper 12 ft of the cross section, and a net flow directed toward the head of the estuary in the layers below this depth. The dominant processes which control the distribution of salt, and hence would control the distribution of any introduced pollutant, are the longitudinal advection term $(wv_1\ \partial_s/\partial x_1)$ and the vertical eddy diffusion term $[\partial/\partial x_3\ (K_3 w\ \partial_s/\partial x_3)]$. Here s, the salt concentration, appears instead of ζ, the pollutant concentration. The vertical advection term contributes a smaller but still significant amount to the balance, while the influence of longitudinal diffusion is negligible. In his treatment of the James River, Pritchard averaged the pertinent equations over a tidal cycle, and further computed the vertical nonadvective flux of salt, which is equal to $K_3\, \partial_s/\partial x_3$. From the observed values of the vertical salinity gradient, it is then possible to compute the variation of the vertical diffusivity K_3 as a function of depth. One such evaluation is given in fig. VIII-3. At both the surface and the bottom the coefficient of vertical eddy diffusion is very nearly zero. The variation of density with depth is also shown in fig. VIII-3, and it is seen that the two maxima in the diffusivity occur in the layers of weak stability (small vertical variation in density); correspondingly, the minimum in the diffusivity curve corresponds to the intermediate depth layers of large stability resulting from a rapid increase in density with depth.

VIII-51. Kent and Pritchard[4] showed that the observed distribution of vertical eddy diffusivity in the James River corresponds to that which would be expected from a generalized mixing length theory of diffusion, provided the effects of vertical stability are included. This study offers the possibility that the vertical eddy diffusivity for a laterally homogeneous tidal waterway may be computed from the observed density distribution, the observed longitudinal velocities, and the physical dimensions of the system.

VIII-52. For tidal waterways, such as the James River, in which the longitudinal diffusion is not an important process, equations VIII-7 and VIII-9 become, when averaged over a tidal cycle,

$$\frac{\partial(wv_1)}{\partial x_1} + \frac{\partial(wv_3)}{\partial x_3} = 0 \qquad \text{(VIII-10)}$$

and

$$\frac{\partial \zeta}{\partial t} = - v_1 \frac{\partial \zeta}{\partial x_1} - v_3 \frac{\partial \zeta}{\partial x_3} + \frac{1}{w} \frac{\partial}{\partial x_3} \left(K_3 w \frac{\partial \zeta}{\partial x_3} \right) \qquad \text{(VIII-11)}$$

These expressions may be utilized in predicting the time-dependent vertical and longitudinal distributions of an introduced contaminant in the following manner. The required observational data are: (a) the net nontidal longitudinal velocities v_1 as a function of longitudinal position x_1 and depth x_3; (b) the physical dimensions of the waterway, i.e. the width w as a function of longitudinal position x_1 and depth x_3; and (c) the time average salinity s over a tidal cycle as a function of x_1, x_3 and of time t. Equation VIII-10 can then be utilized to determine the mean vertical velocities v_3 as a function of depth and longitudinal position. Fig. VIII-4 shows the results of such an evaluation of the vertical velocity as a function of depth for the James River estuary. The vertical velocities are generally too small to be observed directly, although because they enter

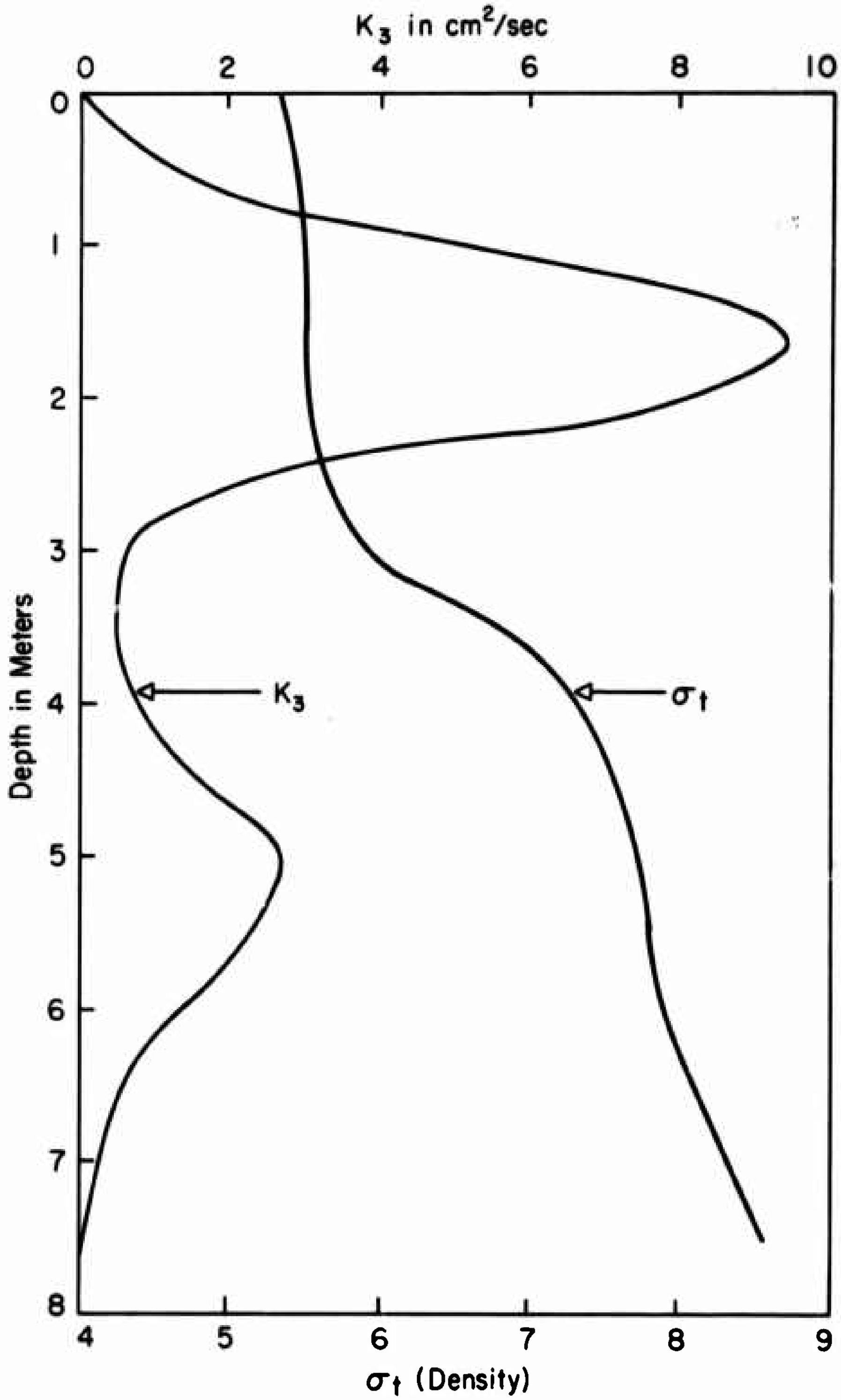


Fig. VIII-3. The vertical diffusivity K_3 as a function of depth for the James River estuary. Also shown is the density expressed in σ_t units. If the density in g/cm^3 is designated by ρ, then $\sigma_t = 10^3 (\rho - 1)$

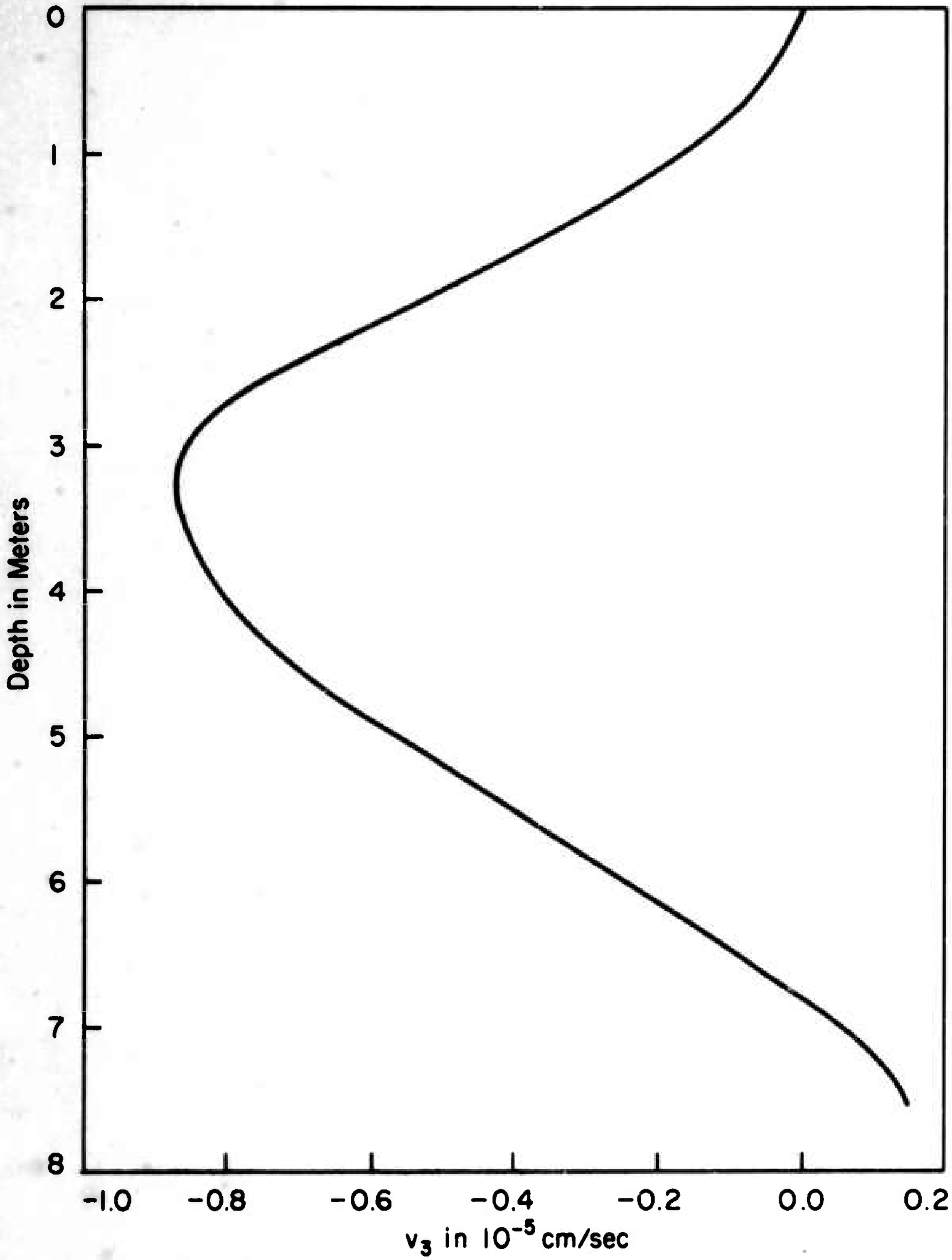


Fig. VIII-4. The vertical velocity v_3 as a function of depth for the James River estuary. Negative values indicate an upward motion

equation VIII-11 in combination with the vertical salinity gradient, which is relatively large, they cannot be neglected. Equation VIII-11, with the salinity s replacing the pollution concentration ζ, can then be employed in obtaining the vertical eddy diffusivity K_3. The integration of equation VIII-11 with respect to the vertical dimension is possible since K_3 can be assumed to be zero at the bottom of the water column. A check on the validity of the approach is provided since K_3 should also be close to zero at the surface.

VIII-53. Once the spatial distribution of K_3 has been obtained, equation VIII-11 may be employed in the numerical determination of the time and space distribution of an introduced pollutant. In making such an evaluation, some initial distribution must be assumed and the numerical integration carried out by a high-speed electronic computer.

One-Dimensional Tidal Waterway

VIII-54. In sectionally homogeneous estuaries, in which the velocity and density distributions are uniform in the vertical and lateral dimensions, the continuity equations may be reduced to the single longitudinal spatial dimension. The equation of mass continuity then becomes:

$$\frac{\partial}{\partial x_1}(\sigma v_1) + \frac{\partial \sigma}{\partial t} = 0 \qquad \text{(VIII-12)}$$

where σ is the area of the cross section at the position x_1. The corresponding equation of pollutant continuity is:

$$\frac{\partial(\sigma\zeta)}{\partial t} = -\frac{\partial(\sigma v_1 \zeta)}{\partial x_1} + \frac{\partial}{\partial x_1}\left(\sigma K_1 \frac{\partial \zeta}{\partial x_1}\right) \qquad \text{(VIII-13)}$$

which, when combined with equation VIII-12, gives as an alternate form:

$$\frac{\partial \zeta}{\partial t} = -v_1 \frac{\partial \zeta}{\partial x_1} + \frac{1}{\sigma}\frac{\partial}{\partial x_1}\left(\sigma K_1 \frac{\partial \zeta}{\partial x_1}\right) \qquad \text{(VIII-14)}$$

Because of their greater simplicity, these equations are the forms of the continuity expressions most frequently employed in studying estuaries. However, it should be clearly appreciated that equations VIII-12, -13, and -14 have real physical significance only when applied to a truly sectionally homogeneous estuary. When utilized in the evaluation of systems in which lateral or vertical variations in density and velocity occur, any physical significance which might otherwise be attached to the diffusivities is lost. Despite this fact, this one-dimensional approach has been quite successful in predicting the gross longitudinal distribution of pollutant for a variety of estuaries.

VIII-55. When equation VIII-12 is averaged over a tidal cycle and integrated from the upstream end of the estuary, at the head of the tide, to any given section in the estuary, we find that:

$$\sigma v_1 = Q_f \qquad \text{(VIII-15)}$$

where Q_f is the volume rate of inflow of fresh water into the estuary. This equation provides the means for determining the net nontidal longitudinal velocity v_1 to be employed in equation VIII-14. If observations of salinity are available for a period when the salinity distribution, neglecting oscillatory variations due to tidal motion, is reasonably steady with time, the longitudinal diffusivity can be computed from the following form of equation VIII-14:

$$\sigma K_1 = \frac{\sigma v_1}{\frac{\partial \ln s}{\partial x_1}} \qquad \text{(VIII-16)}$$

After values of the diffusivity K_1 as a function of position x_1 based on the steady state salinity distribution have thus been obtained, equation VIII-14 can be utilized in computing the time-dependent variations, along the longitudinal axis, of the concentration of an introduced pollutant. Even with this most simple form of the continuity relation it is necessary to use a high-speed electronic computer to integrate equation VIII-14 numerically.

VIII-56. The direct utilization of the longitudinal diffusivity K_1, found from the steady state salinity distribution, in an evaluation of the dispersion of a pollutant is only justified for the period after the pollutant distribution has extended over most of the length of the waterway. It has been found that the magnitude of the diffusivity is directly related to the size of the dispersing volume. During the early stages of dispersion of a locally introduced pollutant volume, the appropriate diffusivities would be much less than the values found from the salinity distribution. This difficulty is overcome by assuming some simple relation between the distance over which the pollutant has spread and the magnitude of the coefficients of eddy diffusion.

VIII-57. Kent[5] developed a different form of equation VIII-14 suitable for numerical integration in the IBM 650 computer. He assumed a linear relation between the size of the dispersing volume and the diffusivity. Designating the diffusivity found from the salt distribution by K_s, the length of the estuary from the mouth to the upper limit of salt intrusion by l_s, the diffusivity applicable to an introduced pollutant by K_ζ, and the longitudinal distance over which the pollutant had spread at any time t by l_ζ, Kent's assumption becomes:

$$K_\zeta = K_s \frac{l_\zeta}{l_s} \tag{VIII-17}$$

Kent tested his assumptions using data obtained in the Delaware estuary model located at the Waterways Experiment Station in Vicksburg, Mississippi. A dye was introduced as an instantaneous release into the model and the movement and dispersion of the contaminated volume was observed for some 50 tidal cycles. Using the observed salinity distribution in the model obtained at the time of the dye studies, Kent computed the longitudinal diffusivities K_s from equation VIII-16. Starting with the observed dye distribution two tidal cycles after release, Kent employed equations VIII-17 and VIII-14 in the numerical determination of the longitudinal distribution of pollutant at later times. Fig. VIII-5 shows the computed and observed dye distribution for the 5th, 10th, 15th, and 20th tidal cycle after release for one of the tests. It is seen that the numerical approach quite closely approximates the observed distributions.

VIII-58. One significant observational feature of the movement and dispersion of the dye introduced into the Delaware model was that the peak concentration did not show a net rate of downstream movement equal to the mean longitudinal velocity v_1, but in fact moved at a significantly slower rate. Thus, the theoretical displacement of a water particle moving with the velocity v_1 advances down the estuary much faster than did the observed position of the peak concentration. On the other hand, the computed position of the peak concentration from the numerical integration of equation VIII-14 closely matches the observed points. An expansion of the second term on the right side of equation VIII-14 leads to the following form of that equation:

$$\frac{\partial \zeta}{\partial t} = -\left[v_1 - \frac{1}{\sigma}\frac{\partial(\sigma K_1)}{\partial x_1}\right]\frac{\partial \zeta}{\partial x_1} + K_1 \frac{\partial^2 \zeta}{\partial x_1^2} \tag{VIII-18}$$

It is seen that the term $1/\sigma\ \partial(\sigma K_1)/\partial x_1$ enters this form of the equation as a correction on the velocity v_1. In this case the diffusivity K_1 increased in the seaward (positive x_1) direction, as did the cross-sectional area σ. Hence, the effective velocity in the continuity equation is less than the actual mean velocity,

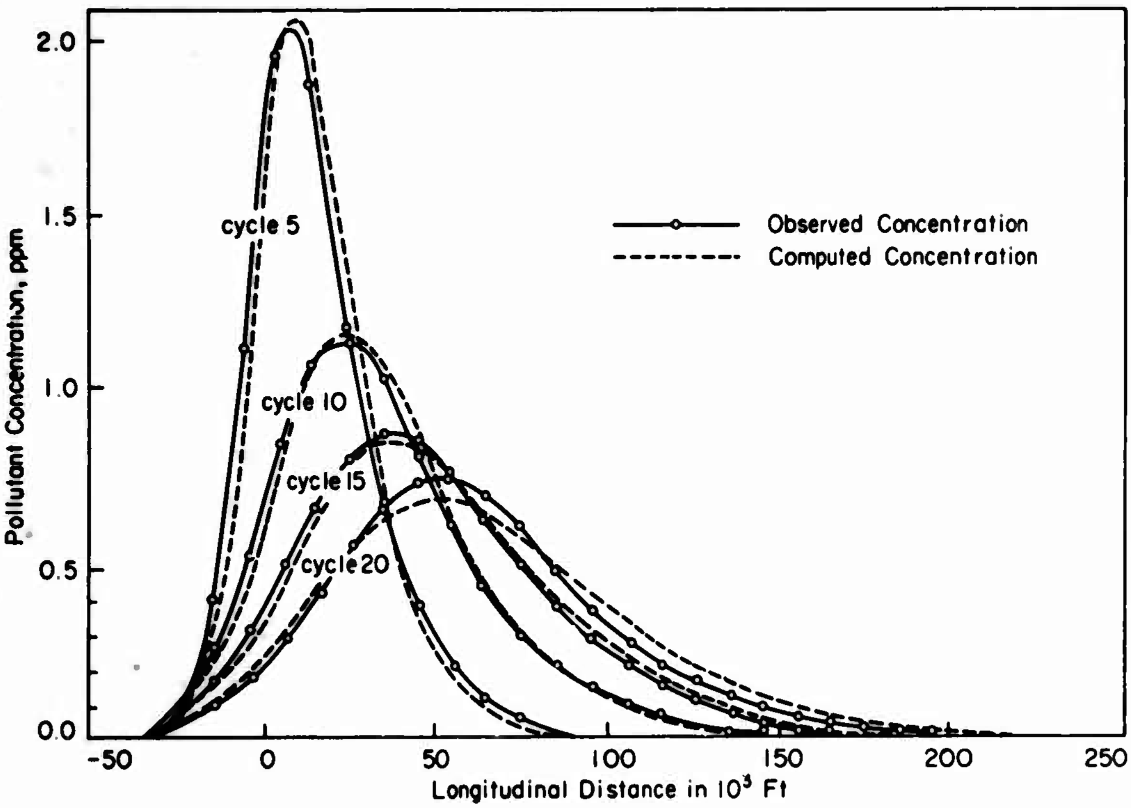


Fig. VIII-5. Observed and computed pollutant concentration as a function of longitudinal distance from release point in Delaware estuary model (after Kent, 1958)

and corresponds to the observed movement of the contaminated volume

VIII-59. Stommel[6] has presented a procedure for computing the distribution of pollutant from a continuous source introduced into a sectionally homogeneous estuary under steady state conditions. The longitudinal eddy diffusivity is determined from the steady state salinity distribution, as described previously. The steady state distribution of pollutant concentration is then found from a solution of the steady state form of equation VIII-13 by the method of relaxation.

VIII-60. If the eddy diffusivities are assumed to be constant in time and space, it is possible to obtain explicit solutions for either an instantaneous point source or a continuous point source in a uniform current field. In this case, the process of diffusion is said to be "Fickian" in character. As has been pointed out in previous paragraphs, the magnitude of the vertical diffusivity differs markedly from the horizontal diffusivities, and is evidently greatly influenced by vertical stability. The two horizontal diffusivities, in regions far removed from physical boundaries, may be taken as equal to each other. However, the magnitude of these horizontal coefficients depends upon the size of the diffusing volume. Hence, Fickian diffusion cannot adequately describe the mixing processes in a tidal waterway.

VIII-61. In recent years a number of theoretical attacks on the problem of horizontal turbulent diffusion in large natural water bodies have been made. These theoretical studies treat the case of the diffusion of an instantaneous vertical line source in a fluid of infinite extent having a uniform horizontal velocity. The initial source is assumed to extend over a depth interval D, and it is further assumed that there is no vertical advective or diffusive flux of material outside of the layer of depth D. Akiro Okubo[7] has made a comprehensive review of these theoretical studies, comparing them to existing data on turbulent diffusion in the sea.

VIII-62. These theories are not applicable in detail to diffusion in estuaries, since the presence of bottom and side boundaries, the nonuniformity of the velocity field, and the vertical variation in density all violate the assumed conditions in the theoretical treatment. However, certain general conclusions based on the theoretical studies are of value in the development of empirical and theoretical studies of turbulent diffusion in tidal waterways.

VIII-63. The existing theories of horizontal turbulent diffusion in the sea can be divided into two groups. One set of solutions predicts that the peak concentration in a diffusing volume of pollutant originally released as a two-dimensional line source will decrease as the inverse second power of time. That is:

$$\zeta p(t) \approx \frac{M}{D \cdot t^2} \qquad \text{(VIII-19)}$$

where

ζp = maximum concentration in the diffusing volume at time t after release of pollutant

M = amount of pollutant introduced

D = vertical thickness of layer containing diffusing volume

The second group of solutions predicts that the peak concentration will decrease as the inverse third power of time. That is:

$$\zeta p(t) \approx \frac{M}{D \cdot t^3} \qquad \text{(VIII-20)}$$

Some of the existing data support the relation shown in equation VIII-19, while some support relation shown in equation VIII-20. Most sets of observations give results intermediate to these two rates of decrease in peak concentration.

VIII-64. The distribution of a pollutant from a continuous discharge may be considered as the sum of the distributions of an infinite number of infinitesimal instantaneous sources discharged in rapid time sequence. Under this assumption, it should be possible to integrate the solutions for an instantaneous source with respect to time to obtain a set of solutions for the continuous discharge case. As a practical matter, not all of the various solutions for horizontal diffusion are amenable to integration. Those that can be integrated all provide the same general form for the distribution of contaminant along the central axis of a pollutant plume extending down-current from a continuous source. The solutions differ with respect to the shape of the concentration distribution across the plume. These solutions predict that the maximum concentration at any distance x from the source along the plume will decrease as the inverse first power of distance. That is:

$$\zeta p(x) \approx \frac{Q}{D \cdot x} \qquad \text{(VIII-21)}$$

where Q is the rate of discharge of pollutant. Observations made under conditions of a relatively steady current field support this prediction. However, if the discharge occurs in a tidal waterway in which the major current field is the oscillatory tidal currents, the plume does not have time to reach the steady conditions required for equation VIII-21 to be valid. On each tidal reversal in current, the existing plume is folded back on itself, and a new plume starts at the source extending in the direction of the then prevailing tidal current. This produces a complex concentration distribution which has not yet been satisfactorily described by theory.

Direct Measurements of the Movement and Diffusion of an Introduced Pollutant in a Tidal Waterway

VIII-65. While relations based on theory or on a combination of theory and empirical information are of use in making preliminary estimates of the probable distribution of pollutants in a tidal waterway under a given set of environmental conditions, such relations are as yet not adequate to give the detailed predictions frequently needed. An alternative approach which in the last few years has proven quite valuable is to simulate the proposed discharge by introducing a tracer material into the waterway and measuring the spatial and temporal distributions of tracer concentrations in the waterway. The technique which has proven most useful involves the use of the rhodamine family of fluorescent dyes as described by Carpenter[8] and by Pritchard and Carpenter.[9] The technique allows the continuous measurement of tracer dye concentration with a routine field detection limit of about 2 parts in 10^{11}. Recent improvements in the sensing instrumentation give promise of routine field measurements in water of low turbidity with detection limits of about one part per trillion (1 part in 10^{12}).

VIII-66. Using this tracer technique, certain implications of the various theoretical attacks on turbulent diffusion have been checked by direct studies in tidal waterways. Fig. VIII-6 shows the observed decrease in peak concentration with time following the instantaneous release of 50 lb of rhodamine B at a position in the Chesapeake Bay. It is noted that this set of observations supports the theoretical prediction that the concentration will decrease as the inverse second power of time. Fig. VIII-7 gives the concentration, as measured along the central axis of a plume of dye, which extends down-current from the location of a continuous discharge, under conditions of a relatively steady current. These data give a good fit to the theoretical prediction that the concentration will decrease as the inverse first power of distance. Fig. VIII-8 shows, however, that when the continuous discharge is made in a tidal waterway with oscillatory currents, the observed down-current concentration distribution does not satisfy the theoretical relation.

VIII-67. Because of the high sensitivity of the rhodamine technique, proposed discharges of relatively large volumes of polluted effluent can be simulated by a very small amount of dye. For example, Pritchard and Carpenter[10] used this technique to estimate the probable distribution of pollutant from three proposed sewage outfalls which would discharge 10 million gallons per day of treated wastes into tidal waters. The experiments involved the continuous release of 3 lb of dye per day for a 30-day period at two of the locations, and 5 lb of dye per day for 10 days at the third location. The predicted pollutant distribution based on one

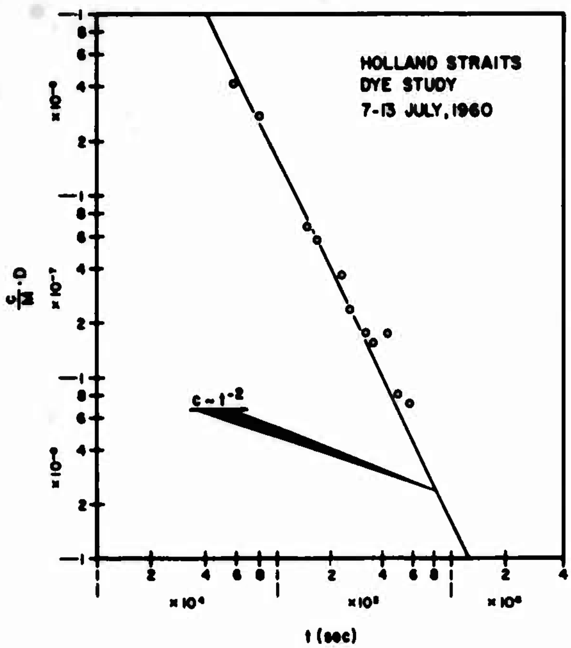


Fig. VIII-6. The decrease in peak concentration with time, as observed from an instantaneous release of tracer dye made in the Chesapeake Bay

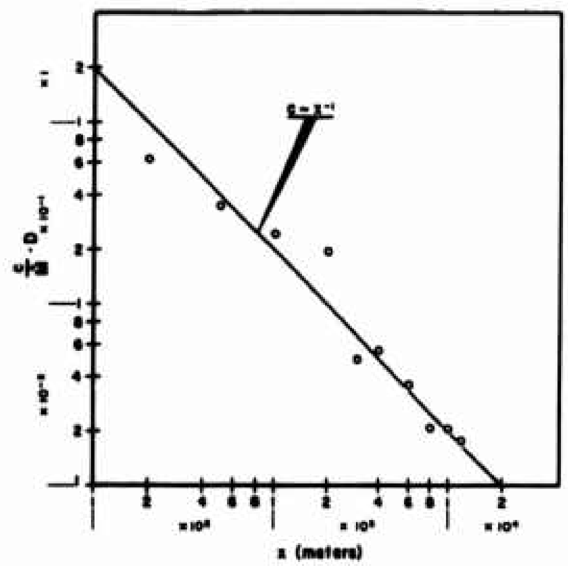
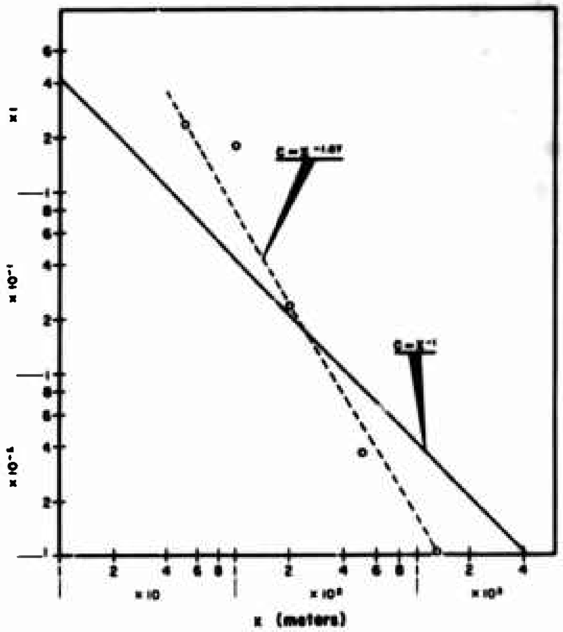


Fig. VIII-7. The decrease in peak concentration with distance along the axis of a plume from a continuous discharge of tracer dye under conditions of a steady current

Fig. VIII-8. The decrease in peak concentration with distance along the axis of a plume from a continuous discharge of tracer dye under conditions of an oscillatory tidal current



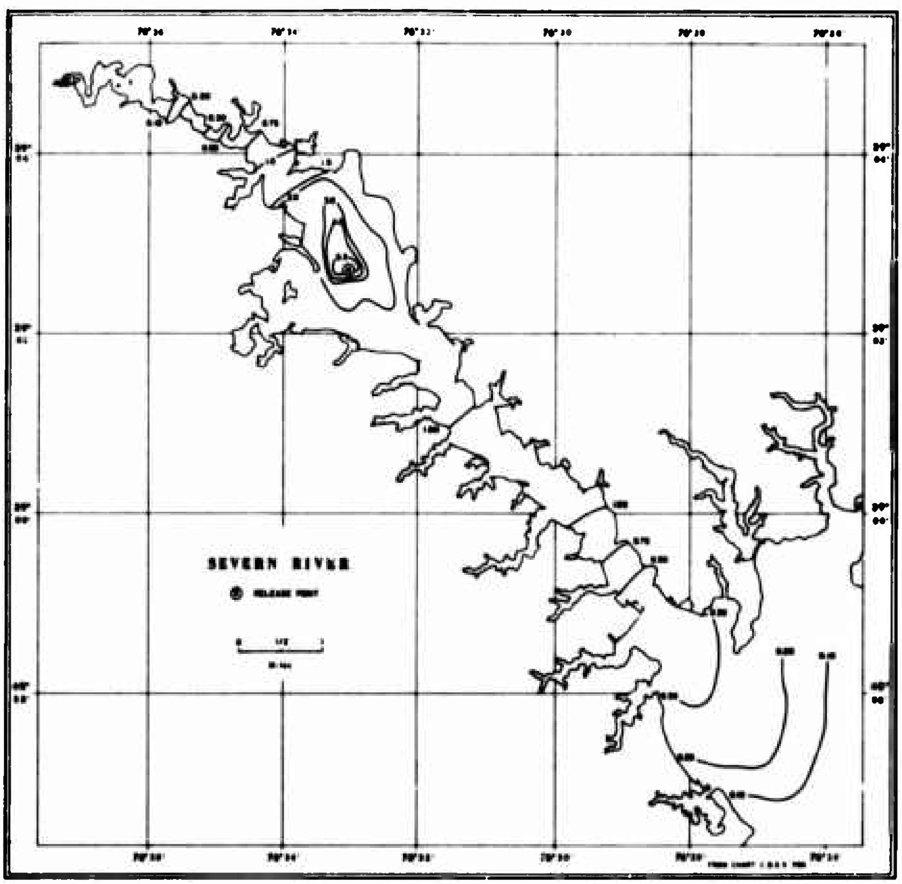


Fig. VIII-9. Horizontal distribution of tracer contaminant in the Severn River estuary during June-July 1963, resulting from a continuous release at a point R approximately 7 miles above the mouth. Contours represent relative contaminant concentration factor C_F in ppm $\times 10^{-3}$ (see text for explanation) of these studies is shown in fig. VIII-9. The proposed outfall sewer was to discharge into the Severn River, a tributary estuary to the Chesapeake Bay. The concentration contours are given in terms of a component of the effluent which would have an initial, undiluted concentration in the waste stream of 1 ppm for a discharge rate of one million gallons per day. To obtain the probable concentrations of any given pollutant component for any proposed discharge rate, it is only necessary to multiply the contours by the product of the undiluted concentration of the component in the waste stream in ppm times the rate of effluent discharge in millions of gallons per day. In these experiments the dye was detectable down to concentrations which would correspond to completely negligible concentrations of the proposed effluent.

The Flushing of Estuaries

VIII-68. The term "flushing" is utilized to denote the exchange of the waters of an estuary or other embayment with the adjacent coastal waters. The same processes which lead to a renewal of water within the waterway would also lead to the flushing or removal of introduced pollutants from the waterway. Thus, in previous paragraphs the dispersion of pollutants within the waterway has been the primary subject of discussion, while now the question of exchange between the waterway and the adjacent coastal waters will be treated.

VIII-69. The flushing of a tidal waterway results from the following processes: (a) tidal exchange, (b) river inflow, (c) density-induced net circulation, and (d) wind-induced currents and meteorological tides. In any particular system one or two of these processes generally dominate the exchange of the waters within the waterway with the adjacent coastal waters. Before considering each of these processes in detail, a clear understanding of the meaning of "exchange" as used here is required.

VIII-70. For the purpose of this preliminary discussion consider the tidal waterway as a tank connected to a much larger tank (the adjacent coastal waters) by two pipes. Suppose one pipe enters the smaller tank near the bottom, and the other pipe enters near the surface. Assume first that water flows from the large tank to the small tank in one pipe, and from the small tank to the large tank in the other pipe. If no mixing occurs between the inflowing waters and the resident waters, then the rate of "renewal" of the waters in the small tank is a simple linear process. Designating the fractional rate of renewal per unit time by γ (i.e. the volume of new water brought in each unit of time divided by the total volume of the tank), then the time interval required to renew or exchange any fraction f of the water resident in the tank at any given time is

$$t_f = \frac{f}{\gamma} \tag{VIII-30}$$

The time required to replace all the water in the small tank (representing the waterway) would be $1/\gamma$. The factor γ is called the exchange coefficient.

VIII-71. Now consider the case where the water entering the small tank is immediately and completely mixed with the water resident in the tank. The water flowing out of the small tank is now composed of a mixture of resident water and of new water. The rate of renewal or exchange of the water in the small tank is no longer a linear one, but rather an exponential process. Theoretically, an infinite time is required to renew all of the water in the tank. The relation between the fraction of the water renewed f and the time interval required for such renewal t_f is:

$$f = 1 - e^{-\gamma t_f} \tag{VIII-31a}$$

or

$$t_f = \frac{1}{\gamma} \ln\left(\frac{1}{1 - f}\right) \tag{VIII-31b}$$

VIII-72. For purposes of comparison of one waterway with another, it is convenient to use a single parameter which expresses the rate at which the waters in the waterway are renewed. One such parameter is called the "flushing time," or the "mean residence time," which simply states the average time that a particle of water remains in the waterway before being replaced by a new particle. For the case of exchange without mixing, the mean residence time is given by equation VIII-30 when f is equal to 0.50. The mean residence time for exchange with complete mixing is given by equation VIII-31b when $\ln[1/(1 - f)]$ is equal to unity. Thus, it is seen that in this case of exponential rate of exchange the mean residence time does not correspond to the time interval required to replace 50 percent of the resident water, but rather the time interval required to replace $1 - 1/e$, or 63 percent of the resident volume.

VIII-73. A second parameter frequently used in describing flushing rates is the "half-life" of the water body, the time interval required to replace 50 percent of the water resident at any given time. For the case of exchange without mixing, the half-life is equal to the mean residence time; that is:

$$t_{1/2} = \frac{0.50}{\gamma} \tag{VIII-32}$$

For the case of exchange with complete mixing, the half-life of the water body has the same connotation as the similar expression for the decay of radioactive material, and equation VIII-31b becomes

$$t_{1/2} = \frac{0.693}{\gamma} \qquad \text{(VIII-33)}$$

Thus, the half-lives for these two extreme cases (no mixing and complete mixing) are more nearly the same than are the corresponding mean residence times.

VIII-74. The processes of exchange in a real water body exhibit properties somewhere between the extreme cases discussed above. There is always some mixing between the new water and the resident water; however, complete mixing is unlikely to occur. Since the half-life is less dependent on the degree of mixing than is the mean residence time, the former parameter is perhaps the more useful single indication of renewal rates.

VIII-75. For the purpose of presenting the manner in which the various processes listed in paragraph VIII-69 influence the flushing of a tidal waterway, consider a coastal embayment having a free connection with the open coastal waters It is assumed that a river enters the landward extreme of the embayment, and that oscillatory tidal motion produces velocities which are locally large compared to those associated with the nontidal circulation. A certain fraction of the volume of the estuary is moved in and out through the mouth of the embayment each tidal cycle as a result of the tidal motion. If no mixing occurred between the coastal waters and waters flowing out of the embayment on ebb tide, or between the waters returning to the embayment on flood tide and the resident estuarine waters, then the tidal oscillations would not contribute to the flushing of the waterway. In general, however, there will be some alongshore movement of coastal waters which will result in replacement of a portion of the estuarine water which flows out of the embayment on ebb tide by new water. The inflow on flood tide also partially mixes with the resident water in the embayment. As a result there is an exchange, each tidal cycle, of resident estuarine water with the coastal water. The exchange coefficient γ associated with this tidal exchange is given by the product of the fraction of the volume of the embayment which is brought into the estuary during the flood phase of the tide times the fraction of this flood tide volume which is new coastal water.

VIII-76. The concept of exchange may be applied to segments of a complex estuarine or embayment system as well as to a simple coastal embayment. The source of replacement water for a tributary estuary or embayment is then the adjacent larger embayment. Baltimore Harbor is an example of a tidal waterway which is tributary to a large estuarine system. The source of the renewal water in this case is the Chesapeake Bay. The volume of water brought into Baltimore Harbor on each flood tide is about 7.5 percent of the total Harbor volume. The circulation pattern in the Chesapeake Bay indicates that about 20 percent of the flood tide volume of the Harbor represents new water which did not leave the Harbor on the previous ebb tide. Thus, the exchange coefficient related to tidal flushing of this waterway is $\gamma = 0.075 \cdot 0.20 = 0.015$. This type of exchange is more nearly exponential than linear in character, and hence the half-life would be 46 days as given by equation VIII-33. The ratio of flood tide volume to total volume will differ from waterway to waterway, as will the fraction of the flood tide volume which is new water. However, it has been observed that the latter fraction frequently falls between 0.1 and 0.2. As will be evident from the discussion which follows, other processes are usually more significant in flushing an estuary than the tidal exchange.

VIII-77. The net freshwater inflow into the waterway, due to river inflow and the excess of land drainage and direct precipitation over loss by evaporation, can be considered as providing a second mechanism for flushing the estuary. This net inflow must pass on through the waterway and out to sea, since the mean water level of the waterway does not change materially with time. In most cases the direct effect of freshwater inflow on the flushing of a tidal waterway is relatively unimportant. Of much greater significance is the indirect effect of the freshwater inflow through the contribution this fresh water makes to the density distribution, and hence to the density-induced net circulation pattern.

VIII-78. If the estuary under consideration is a partially mixed estuary with a net circulation pattern exhibiting a net seaward-directed flow in the upper layers and a net flow directed up the estuary in the deeper layers, the flushing rate of the estuary will be much larger than can be accounted for by the mechanisms of tidal exchange or direct influence of freshwater inflow. A rather simple expression can be developed for the net volume rate of outflow and inflow for the case where longitudinal diffusion is unimportant. Designating the volume rate of inflow from the river as r, the direct precipitation plus local land drainage as P, and the evaporation from the surface of the waterway as E, then the net freshwater inflow R is given by

$$R = r + P - E \qquad \text{(VIII-34)}$$

Designating the mean salinity of the upper seaward-flowing layers by s_u, the volume rate of net seaward-directed flow of these layers by V_u, the salinity of the lower layer having a net flow directed up the estuary as s_ℓ, and the volume rate of the flow in these layers by V_ℓ (see sketch), it can be shown that

$$V_u - V_\ell = R \qquad \text{(VIII-35)}$$

and

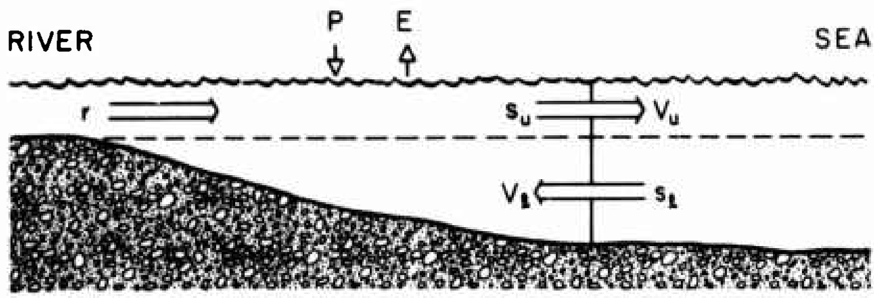
$$V_u s_u - V_\ell s_\ell = 0 \qquad \text{(VIII-36)}$$

These expressions lead to the relations

$$V_u = R \frac{s_\ell}{s_\ell - s_u} \qquad \text{(VIII-37a)}$$

and

$$V_\ell = R \frac{s_u}{s_\ell - s_u} \qquad \text{(VIII-37b)}$$



VIII-79. The James River in Virginia is an example of a partially mixed estuary having a net circulation pattern similar to the one indicated in the previous paragraphs and within which longitudinal diffusion is not significant. Direct current measurements in the James estuary have confirmed the validity of the simple expressions given by equation VIII-37 over much of the length of this waterway. Near the upper extreme of saltwater intrusion, longitudinal mixing becomes important, and the equations no longer apply. Fig. VIII-10 is a schematic presentation of the salinity distribution and net flow pattern in an estuary such as the James River. In this schematic longitudinal section, it is assumed for simplicity that direct precipitation is balanced by evaporation. The freshwater inflow R is then the river flow. In this particular case the difference between the mean salinity of the surface layers and that of the lower layers is taken as uniformly 3 parts per thousand over the length of the estuary, except for the upper reach of the estuary where the surface and bottom salinity both approach zero. At the mouth of the estuary, where the salinity of the surface layers is taken as 27 parts per thousand, the volume rate of outflow through the upper portion of the cross section is ten times the volume rate of river inflow. Mass continuity then requires that the net inflow along the bottom equal nine times the river flow, and also requires

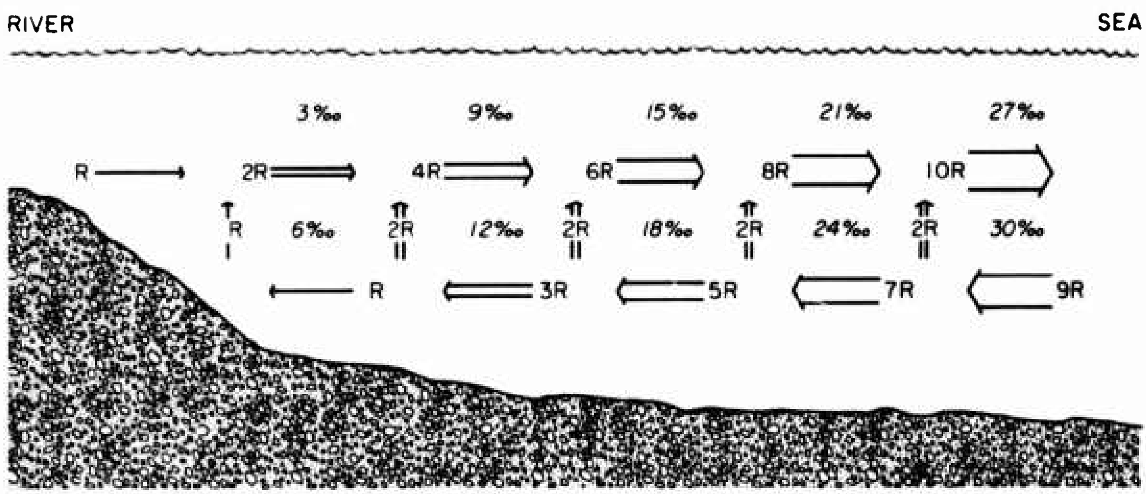


Fig. VIII-10. Schematic flow pattern and salinity distribution (parts per thousand) in a partially mixed estuary. Volume rate of flow is expressed in terms of the volume rate of river flow R that throughout the length of the estuary net vertical motion take place from the lower layers to the upper layers.

VIII-80. Within the estuary, some vertical mixing occurs between the counterflowing layers. In the coastal waters outside the mouth of the estuary, the outflow of the surface layers of the estuary frequently curves and becomes parallel and close into the coastline, while the origin of the deeper inflowing layers of the estuary is from the coastal waters directly out from the mouth. The volume rate of net outflow in the upper layers at the mouth then may be taken as the rate at which net circulation brings about replacement of the estuarine waters. In the schematic example shown in fig. VIII-10, the rate of flushing of the estuary would be ten times the rate which would just be required to remove the inflowing fresh water.

VIII-81. Baltimore Harbor is an example of a small tributary embayment to a larger estuarine system, within which the local freshwater inflow has much less effect on the density distribution than does the salinity distribution in the waters of the adjacent main estuary. The three-layered circulation pattern characteristic of this tributary embayment was described in paragraphs VIII-35 through VIII-38. The point to be made here is that this net nontidal circulation pattern is by far the most effective mechanism for renewal of the Harbor volume. The exchange coefficient for the Harbor due to this mechanism is about 10 percent per day, while as pointed out in paragraph VIII-76, the exchange coefficient due to tidal exchange is only about 1.5 percent per day. The half-life of the Harbor associated with the net flow pattern is then about 7 days, while that associated with tidal exchange is about 46 days. For comparison, the time required for the average local freshwater inflow to replace half the volume of the Harbor is about 240 days.

VIII-82. Paragraphs VIII-39 and VIII-40 describe the circulation pattern for the Magothy, a tributary embayment for which the mechanism driving the circulation pattern is associated with the difference in the time rate of change within the embayment as compared to the adjacent waters of the Chesapeake Bay. For such an embayment, it is possible to obtain an expression for the exchange coefficient based on some simplifying assumptions regarding the processes which lead to the rate of change of salinity within the tributary embayment. For this purpose designate:

S_t = mean salinity within the tributary embayment at time t

S_{t+dt} = mean salinity within the tributary embayment at time $t + dt$

S_b = mean salinity of the adjacent Bay waters

Assuming that the change in salinity within the tributary embayment during the time interval dt results from exchange of waters from within the embayment for waters from the adjacent larger estuarine system, then, for a unit volume

$$S_{t+dt} = nS_t + (1 - n) S_b$$

where n represents the fraction of the unit volume which originated within the tributary embayment and (1 - n) represents that fraction of the mixture which came from the adjacent Bay. Since, approximately,

$$S_{t+dt} = S_t + \frac{\partial s}{\partial t} dt$$

and since the ratio 1 - n/dt is the fractional rate of addition of new water to the embayment, i.e. simply the exchange coefficient γ, then

$$\frac{\partial s}{\partial t} = \gamma (S_b - S_t) \qquad \text{(VIII-38)}$$

VIII-83. Observations made in the Magothy estuary and in the adjacent Chesapeake Bay can be employed in a sample computation using equation VIII-37. During the spring of 1958, the salinity in the Magothy showed a rate of change of -0.15 parts per thousand per day during a time when the average difference between the salinities of the adjacent Bay waters and the Magothy waters was -1.60 parts per thousand. The exchange coefficient was then given by $\gamma = 0.15/1.60 = 0.094$ per day. The half-life of the Magothy then would be between that given by equation VIII-32:

$$t_{1/2} = \frac{0.50}{0.094} = 5.3 \text{ days}$$

for the case of no mixing, and that given by equation VIII-33:

$$t_{1/2} = \frac{0.693}{0.094} = 7.4 \text{ days}$$

for the case of complete mixing. A computation of the exchange coefficient due to tidal exchange alone gives $\gamma = 0.02$, which corresponds to a half-life, by equation VIII-33, of 35 days. The net nontidal circulation is thus a more effective mechanism than tidal exchange for flushing of this type of tidal waterway, just as in the several other types of estuaries discussed in previous paragraphs.

VIII-84. Wind-induced currents can contribute to the rapid replacement of the volume of a tidal waterway. A wind blowing down an embayment toward the ocean produces a surface flow directed out of the embayment having a velocity of about 2 percent of the wind speed. A counterflow will occur in the deeper waters directed into the embayment. Wind blowing from the coastal waters into the embayment will produce an opposite circulation pattern. It would appear that this wind-induced circulation pattern should be able to effectively flush a small embayment in a relatively short time. However, observational data sufficient to determine flushing rates due to wind-induced motion have not been obtained.

VIII-85. Meteorological tides can also produce rapid exchange of waters of small coastal embayments. These water level changes occasionally represent changes in volume of the embayment of 30 to 50 percent. Neither the wind-induced circulation pattern nor meteorological tides can be counted on as regular flushing mechanisms, since their occurrence is infrequent and unpredictable.

VIII-86. The rhodamine tracer technique may be employed to directly measure the exchange rates for small and moderately sized tidal waterways. For example, at the end of the 30-day period of pumping of tracer dye into the Severn River estuary, in connection with the study discussed in paragraph VIII-67, approximately 100 lb of dye was distributed through the length of the estuary. After the introduction of dye was stopped, the distribution of dye concentration continued to be monitored over the next 90 days. During this time, dye was flushed from the Severn into the adjacent Chesapeake Bay. This flushing rate was computed by integrating the observed dye concentration distribution over the volume of the estuary, and thus determining the total amount of dye remaining in the estuary after specified intervals of time. By this means it was determined that the renewal rate of the Severn estuary is quite low, on the order of 2.5 percent per day.

VIII-87. Most pollution problems are more critically related to the detailed local distribution of pollutant concentration than to the mean pollutant concentration in the waterway. While the overall rate of exchange of the waters of a given estuary or embayment with an adjacent, much larger water body certainly affects the local buildup of pollutant concentration, there is a tendency to be concerned with the prediction of the probable steady state concentration distribution for a particular waste discharge, rather than simply determining the overall exchange characteristics of the waterway. The use of such general flushing parameters as the exchange ratio, the flushing time, and the half-life are generally restricted to purposes of gross comparison of different estuarine systems.

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CHAPTER IX

HYDRAULIC MODEL STUDIES OF TIDAL WATERWAYS PROBLEMS

by

H. B. Simmons and C. P. Lindner

Introduction

IX-1. Models have proven useful in all branches of hydraulic science for obtaining information that cannot be derived by rigid analysis or with sufficient accuracy by rationalization. As their potentialities have become generally known, they have been used with increasing frequency. In the field of tidal hydraulics, the widespread use of models was probably delayed because of the complicated problems involved and the relatively high cost of constructing and operating appropriate models. The techniques for instrumenting and operating tidal models for accurate reproduction of certain of the phenomena involved had to be worked out. Some models required the use of both salt water and fresh water at the same time, and the model laws for such cases had to be established. The tides had to be reproduced with a high degree of accuracy, which required the development of complex and dependable tide generators. Often sediment had to be introduced to reproduce observed patterns and to study deposition under modified conditions. Suitable model sediments had to be located and appropriate techniques developed for preparation and injection of the sediment. Verification of tidal models is in itself a tremendous task, requiring the procurement and analysis of a large amount of field data, much of which is difficult to obtain with sufficient precision. This applies especially to measurement of current velocities and directions, salinities, and shoaling patterns under ever-changing conditions of tide and freshwater inflow.

IX-2. Without doubt the preceding factors were responsible for delays in undertaking model experimentation in several fields of tidal hydraulics. However, as knowledge of model work expanded, new instruments and techniques were developed, and confidence in the securing of worthwhile results from tidal models increased. With continued development of the country, which required the progressive deepening of harbor channels, tidal problems became increasingly complex and the need for solving them became more urgent. The design of facilities for modern harbors and access channels must take into consideration the dispersion, diffusion, and flushing of wastes discharged purposely or accidentally into the waterways; the fish and wildlife resources of the area;

ground and surface water resources; and many other factors of major importance to our modern civilization. It is no longer permissible to effect physical changes in tidal waterways in the interest of navigation without a comprehensive knowledge of the effects of such changes on all other interests involved. As a result, many models have been built in recent years that have aided materially in predicting the effects of proposed works, and in designing channels and other engineering works to minimize shoaling, to ameliorate or prevent dangerous currents, to control saltwater intrusion, and to solve many other problems peculiar to tidal waterways. They have proved of such value that, for every proposed tidal waterway improvement of substantial cost, a hydraulic model should be used to explore all significant aspects prior to construction of the improvement.

IX-3. Most studies of major problems or proposed improvements in tidal waterways are combined prototype and model studies, and the conclusions are arrived at from examination and analysis of data obtained from both prototype and model. Model results often require interpretation by the use of general knowledge and the intelligent correlation of the data obtained from the prototype. Deduction from the evidence presented often leads to the conclusion that more than one technical solution exists for a given problem, so it is then necessary to determine the most desirable solution. Usually no simple plotting or tabulation of model and prototype data will point to the most desirable solution; however, a thorough analysis of adequate data from both model and prototype, supported by economic and other considerations, will usually lead to selection of the optimum solution.

IX-4. While the hydraulic model provides an excellent means for integrating the numerous forces and factors that affect conditions in tidal waterways, like any tool, the model is subject to certain limitations that must be fully recognized and understood by engineers and scientists who make use of such models. It is the purpose of this chapter to discuss the potential uses and limitations of hydraulic models for studies involving the principal problems encountered in estuaries. These problems include (a) shoaling of entrance channels, interior harbor channels, turning basins, and other navigation facilities and means of reducing this shoaling or minimizing the cost of shoal removal, (b) saltwater intrusion, and especially the effects of proposed changes in the physical and hydraulic regimens of estuaries, (c) the dispersion, diffusion, and flushing of wastes discharged into estuaries, (d) the hydraulics of estuaries as related to the location and design of channels suitable for navigation, (e) flooding by hurricane surges and other unusual tidal phenomena, and (f) proper disposal of dredge spoil removed from channels and other navigation facilities. The authors have attempted to present the information contained herein in such manner that the engineer in the field will be able to evaluate the contributions of hydraulic models in arriving at a satisfactory solution to the particular problem with which he is concerned. To this end, the subsequent parts of this chapter present typical examples of model investigations which involved, as their principal purpose, each of the major problems cited in this paragraph.

Shoaling of Channels and Other Navigation Facilities

IX-5. Models for investigation of plans to reduce shoaling of entrance or interior harbor channels may be either of the fixed-bed or the movable-bed type, depending largely on the nature of the shoaling material involved and the forces which affect transportation and deposition of this material. Movable-bed models are commonly used if the shoaling material is sand and its movement and deposition are affected by wave action as well as tidal and density forces. In movable-bed models, periodic hydrographic surveys and dredging data are used as the basis for determining the long-term trend of change in bed and bank configurations, as well as the shoaling rates and patterns in navigation channels, and these data, together with hydraulic and meteorological data from the prototype, are used for verification of bed movement in the model. Verification of a movable-bed model is considered to have been attained when the model reproduces, within acceptable limits, the changes in bed scour and fill which are shown by prototype surveys and dredging data to have occurred in the period selected for verification purposes. This verification process also establishes the empirical time scale for bed movement and deposition, which cannot be determined mathematically from the linear scale relations of the model. For example, if 10 hr actual operating time is required for the model to reproduce known changes in prototype bed configurations which occurred in a 2-yr period, then the empirical time scale for bed movement in that model is 5 hr model equals 1 yr prototype, or approximately 1:1750.

IX-6. The Absecon Inlet model study, which was carried out by the U. S. Army Engineer Waterways Experiment Station (WES) in the early 1940's, is a typical example of use of a movable-bed model for a study of plans to reduce shoaling of an entrance channel. Absecon Inlet (fig. IX-1) is located at Atlantic City, N. J., and at the time of the model study a navigation channel 20 ft deep at mean low water (mlw) and 400 ft wide was maintained through the inlet to the port facilities at Atlantic City. No jetties had been constructed at the inlet at that time, and dredging to maintain the entrance channel averaged about 365,000 cu yd per yr.

IX-7. The model reproduced the area shown in fig. IX-1 to linear scales

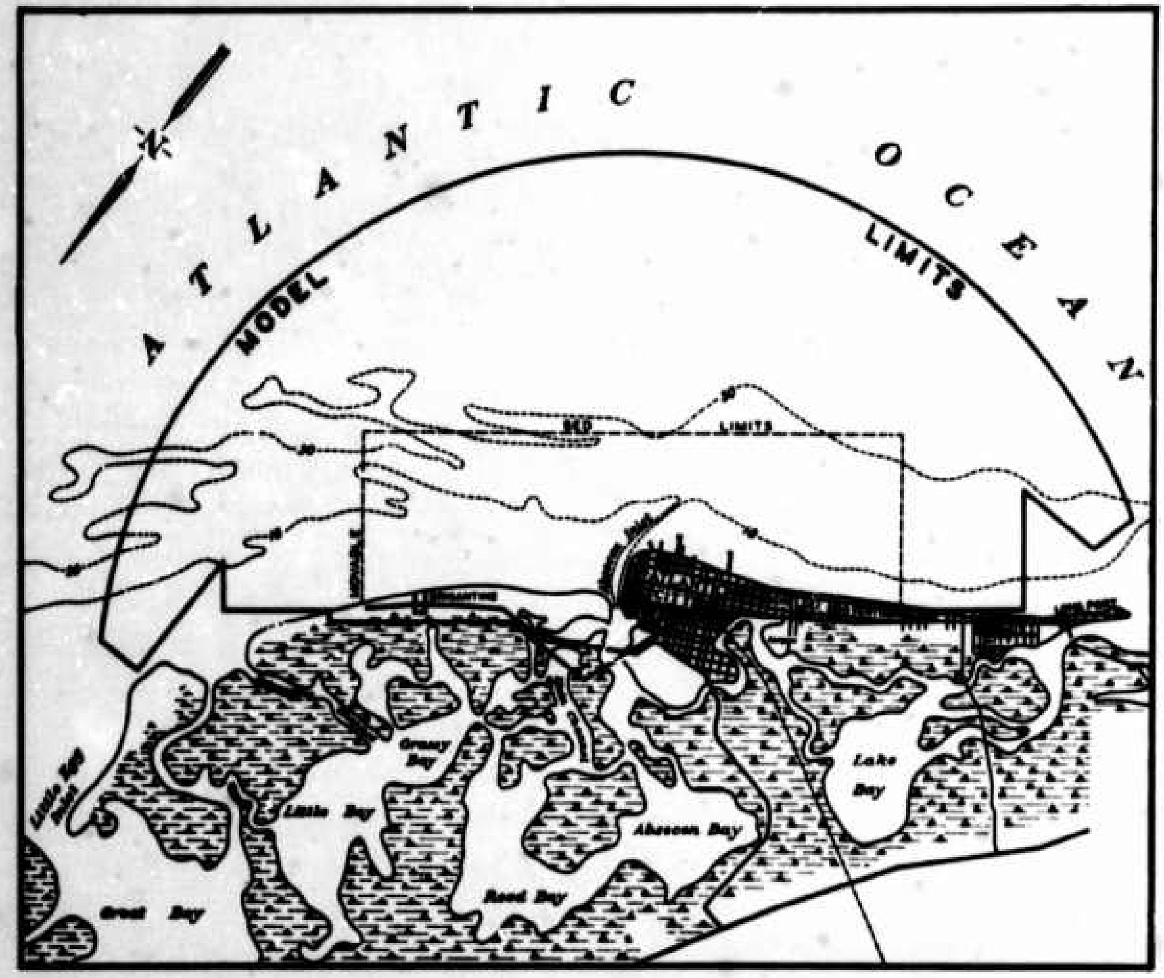

Fig. IX-1. Limits of Absecon Inlet model

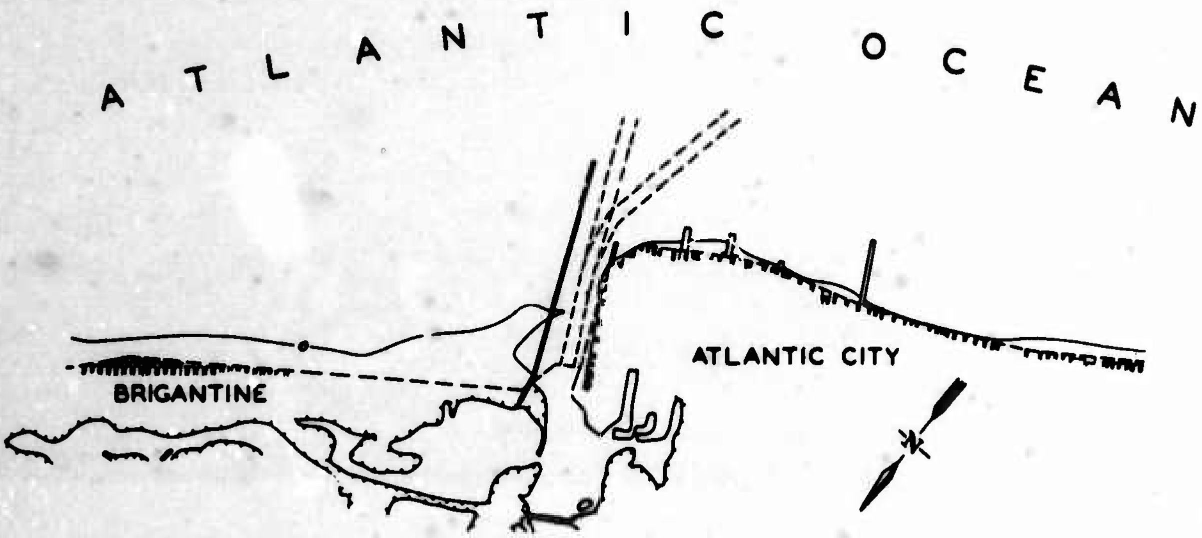

Fig. IX-2. Model jetty plan for Absecon Inlet, New Jersey of 1:500 horizontally and 1:100 vertically. Tides and tidal currents were reproduced by a primary tide generator located in the Atlantic Ocean portion of the model and a secondary tide generator located at the model limit of Absecon Inlet, and waves were reproduced by a wave generator in the model ocean. The prototype period January 1936 to January 1939 was selected for the movable-bed verification of the model. The movable bed was first molded of sand to conform to the prototype hydrographic survey of January 1936, and various operation schedules based on different assumed time scales for bed movement, were followed until, for the final schedule, the conditions of the model bed checked those of the January 1939 prototype survey very closely. It was found that 39 hr of actual model operation were required to accomplish the change in bed conditions between these two surveys, so the empirical time scale for bed changes worked out to be 13 hr model to 1 yr prototype, or about 1:674. All dredging and spoiling operations accomplished in the prototype during the verification period were simulated in the model to time scale. In the prototype, a total of 1,715,000 cu yd of sand was dredged from the entrance channel during the verification period, while in the model it was found necessary to dredge a total of 1,583,000 cu yd (prototype) from the navigation channel to maintain the project dimensions of the channel. Therefore, in addition to reproducing the overall changes in prototype bed configurations during the verification period, the model also reproduced within about 8 percent the prototype shoaling rate of the navigation channel which occurred during the same period.

IX-8. A number of proposed improvement plans were then tested in the Absecon Inlet model, and special attention was paid to their effects on shoaling of the entrance channel and on the extremely valuable recreation beaches at Atlantic City. The results of the model tests indicated that a plan involving the jetty system shown in fig. IX-2 would reduce entrance channel shoaling by about 55 percent and would have no detrimental effects on the beaches. The inshore 1000 ft of the north jetty was at elevation +8.0 ft mlw, the next 250 ft sloped from +8.0 to 0.0, the next 4450 ft was at 0.0, and the next 2300 ft was at +12.0 ft mlw. This plan has not been constructed in nature, but a modification is presently (1964) under construction.

IX-9. A model study of Charleston Harbor, which was conducted at WES during the period 1947 through 1953, is a good example of a fixed-bed model which was used to study plans for reducing shoaling of inner harbor navigation channels. Charleston Harbor (see fig. IX-3) is formed by the Ashley, Cooper, and Wando Rivers. The main navigation channel, which is maintained at a depth of 35 ft at mlw, enters the harbor between two jetties and terminates in Cooper River near Goose Creek. In addition there is an alternate ship channel west of

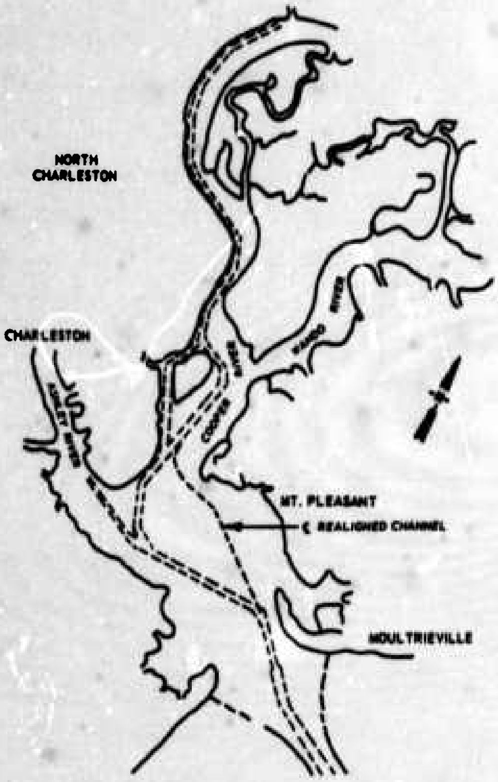

Fig. IX-3. Location map of Charleston Harbor

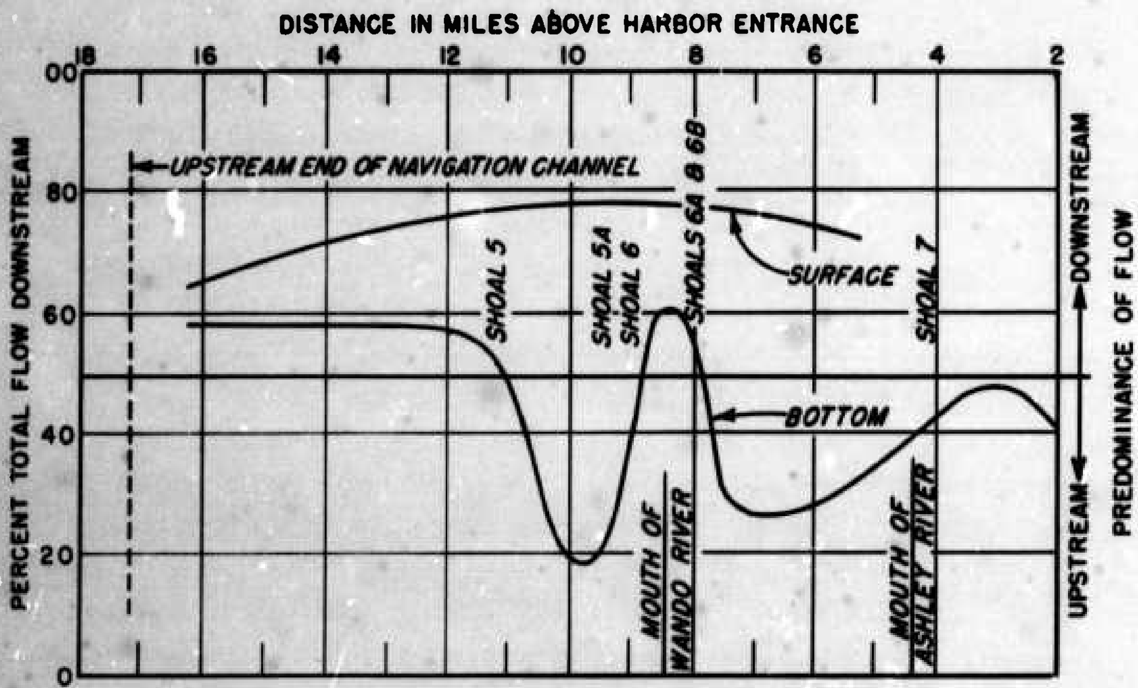

Fig. IX-4. Predominance of flow in Charleston Harbor

Drum Island, a deepwater channel in Shipyard River, and channels of lesser depth in Town Creek and Ashley River.

IX-10. Prior to about 1940, the average freshwater discharge into Charleston Harbor was less than 100 cfs, and the average annual dredging required to maintain a navigation channel 30 ft deep at mlw was of the order of 80,000 cu yd. A project depth of 35 ft at mlw was adopted about that time, but the only dredging required to obtain that channel consisted of dredging to a 35-ft depth over a few crossings between deep pools. In about 1941, the Santee-Cooper Hydroelectric Power Project was placed in operation, which increased the average freshwater discharge into the harbor from 100 cfs to more than 15,000 cfs; this large increase was caused by the diversion of water from the Santee River through the Pinopolis Reservoir and powerhouse into the upper Cooper River and thence through Charleston Harbor to the ocean.

IX-11. By about 1947, when the model study was initiated, there were seven well-defined shoal areas in Charleston Harbor, and the average annual shoaling rate was in excess of 5,000,000 cu yd. It is interesting to note that the area covered by shoal 7, which required dredging in excess of 1,000,000 cu yd annually to maintain a 35-ft depth in 1947, had stable natural depths in excess of -50 ft mlw prior to 1941. Analysis of periodic hydrographic surveys, hydraulic data, and dredging records indicated that the Santee-Cooper Project was responsible for the large increase in shoaling in Charleston Harbor for two principal reasons. Firstly, the diversion of 15,000 cfs of Santee River water into the harbor created a major source of upland sediments, which source did not exist before the diversion was effected and in addition this flow may have induced erosion in the diversion canal and upper Cooper River. Secondly, this large inflow of fresh water changed Charleston Harbor from essentially a saltwater embayment to a partly mixed estuary. In the latter, bottom upstream currents predominate over bottom downstream currents from the harbor mouth to the vicinity of the upstream limits of saltwater penetration. Thus, much of the sediment entrained in the bottom stratum is trapped and cannot be carried to sea by the natural forces.

IX-12. The distributions of flow over a tidal cycle at surface and bottom throughout Charleston Harbor for conditions of mean tide and average freshwater discharge are shown in fig. IX-4. It will be noted that flow in the surface stratum is predominantly downstream throughout the full length of the harbor. However, in the bottom stratum, flow is predominantly upstream from the ocean to about the Charleston Navy Yard (shoal 5), with the exception of one short reach at the mouth of the Ashley River where there is little predominance in either direction, and a second short reach at the mouth of the Wando River. These two rivers carry large tidal discharges but contribute essentially no fresh water to the harbor, and it appears that the turbulence created at their confluences with the harbor proper is sufficient to disrupt locally the vertical salinity gradients which are responsible for the vertical circulation pattern which produces bottom upstream flow predominance elsewhere in the harbor.

IX-13. The relation between the locations of shoals which existed in Charleston Harbor at the time of the model study, and the predominance of flow in the bottom stratum throughout the harbor at that time, is also shown in fig. IX-4. Shoal 7, the largest shoal in the harbor, was located in the area where the upstream bottom flow predominance was interrupted locally at the mouth of the Ashley River, so that sediments were trapped in the region where the upstream and downstream flows were in balance. A similar situation existed at the mouth of the Wando River, and shoal 6, the second largest shoal in the harbor, was found at this location. Shoal 5A, the third largest shoal, was actually a continuation of shoal 6 and existed for the same basic reasons. The remaining smaller shoals upstream from shoal 5 were located in channel reaches having excessive cross-sectional areas and/or unusual flow characteristics, and their locations were apparently not related directly to the bottom flow predominance patterns of the harbor.

IX-14. The Charleston Harbor model reproduced the area shown in fig. IX-3 plus an additional portion of the Atlantic Ocean adjacent to the harbor entrance and the remaining upstream tidal portion of the Cooper River to the Santee-Cooper powerhouse, to linear scales of 1:800 horizontally and 1:80 vertically. Tides and tidal currents were reproduced by a primary tide generator located in the ocean portion of the model and a secondary tide generator located at the model limit of the Wando River. The salinity of the model area was reproduced to the salinity scale of 1:1, and fresh water was introduced at the Santee-Cooper powerhouse to represent the discharge through this structure. Shoaling verification tests were made to select a model sediment which would move and deposit under the influence of the model forces in the same manner that the prototype sediments moved and deposited under the influence of prototype forces. The results of these tests showed that crushed and graded gilsonite, having a specific gravity of about 1.04, would meet these requirements. This material, when injected into the model and allowed to be distributed by the model currents, was found to deposit in a pattern conforming to the known shoaling pattern of the harbor.

IX-15. Tests in the Charleston Harbor model showed that the only real solution to the Charleston Harbor shoaling problem would involve rediversion of the freshwater discharge which created the problem. However, since it was recognized that rediversion of the fresh water would be very costly and would require a great amount of additional study, tests were made to determine whether or not interim improvements at major shoal areas would be technically and economically feasible. For the shoal 7 area, model tests indicated that relocation of the navigation channel to the north side of the estuary (fig. IX-3) would bypass the area in which upstream predominance of bottom flow was interrupted, so that in the relocated channel, bottom flow would be predominantly upstream throughout its full length. The tests indicated that shoaling of the relocated channel would be at least 85 percent less than that in the existing channel through shoal 7. This relocation was effected in 1957, and no maintenance dredging of the relocated channel has been required since that date. In this period, shoaling of the abandoned channel has progressed at about the rate at which it formerly shoaled, so that a tremendous amount of dredging has been saved by the channel relocation. Interim improvement plans for shoals 6 and 5A were also developed in the model and subsequently constructed in the prototype, but these improvements have not been in existence for a sufficient period of time to determine whether or not their effects on shoaling are as predicted by the model tests.

Saltwater Intrusion

IX-16. A model study of Vermilion Bay, Louisiana, which was conducted at WES for the Louisiana Department of Public Works in the period December 1955 through December 1956, is a good example of use of a hydraulic model to study saltwater intrusion. Vermilion Bay (see fig. IX-5) is located on the Louisiana coast between Morgan City and Lake Charles. The bay has a deep and narrow connection to the Gulf of Mexico through Southwest Pass on the south, and a second wide and shallow connection through West and East Cote Blanche Bays and Atchafalaya Bay to the east and southeast. The Vermilion River, which is a relatively small stream, discharges into the northeast side of Vermilion Bay, and the Atchafalaya River, which carries a large freshwater discharge at all times, and Wax Lake Outlet discharge into Atchafalaya Bay.

IX-17. The Vermilion River is used extensively as a source of water for irrigating rice. During dry seasons, when irrigation water is used most extensively, the rate of pumping from the Vermilion River often exceeds the river flow, and salt water from Vermilion Bay moves rapidly upstream and eventually reaches the pump intakes. Considerable damage to the rice crop may then result, either from lack of water needed by the rice if pumping is curtailed, or from the salinity of the water pumped into the rice fields if pumping is continued. Historical salinity data show that the entire bay complex is freshened during the high discharge season on the Atchafalaya River, which usually extends from about February through June. Following the high discharge period, salinities throughout the bays increase gradually, usually reaching a maximum in about September or October, which also happens to fall within the period of high use of irrigation water. These data also indicate that the primary source of salt water into Vermilion Bay is through Southwest Pass, with a secondary source through Atchafalaya Bay and East and West Cote Blanche Bays. It was therefore considered possible that closure of Southwest Pass would reduce the rate of influx of salt water into Vermilion Bay to such extent that pumping from the Vermilion River could be continued for an appreciably longer period of time than is now possible.

IX-18. The Vermilion Bay model reproduced a portion of the Gulf of Mexico, all of Atchafalaya Bay, East and West Cote Blanche Bays, Vermilion Bay, the lower reaches of the Atchafalaya and Vermilion Rivers, and the lower reaches of other streams which contribute fresh water to the bay complex. The model reproduced the approximate area shown in fig. IX-5 to linear scales of 1:2000 horizontally and 1:100 vertically. Tides were reproduced by a tide generator located in the Gulf of Mexico portion of the model, which also contained provisions for reproducing littoral or alongshore currents from either direction. All freshwater tributaries were equipped with weirs for metering freshwater inflow, and the model was operated with the Gulf of Mexico portion filled with salt water to the salinity scale of 1:1. Because the entire bay complex is quite shallow, usually less than 12 ft in depth, surface wind waves play a significant role in the vertical mixing of salt water and fresh water therein, and this effect had to be reproduced in the model to reproduce accurately the salinity regimen of the prototype. Since it was not feasible to reproduce the wind waves in the distorted model, their mixing effects were simulated by means of oscillating fans positioned to blow in a random pattern on the model water surface. Fans were added as necessary until vertical salinity gradients throughout the model were in good agreement with those observed in the prototype for model verification purposes.

IX-19. The salinity verification of the Vermilion Bay model consisted of reproducing to proper time and discharge scales the measured freshwater inflow of all significant tributaries for a 1-yr period, during which surface and bottom prototype salinity data for a great number of stations throughout the bay complex were available for checking against salinities at corresponding stations in the model. This verification showed that the salinity regimen of the model was in good agreement with that of the prototype throughout the entire period involved, so the model was considered to be ready for testing the proposed plan for closing Southwest Pass. Two conditions of freshwater inflow were selected for test purposes. The first made use of prototype freshwater inflow data for the water year

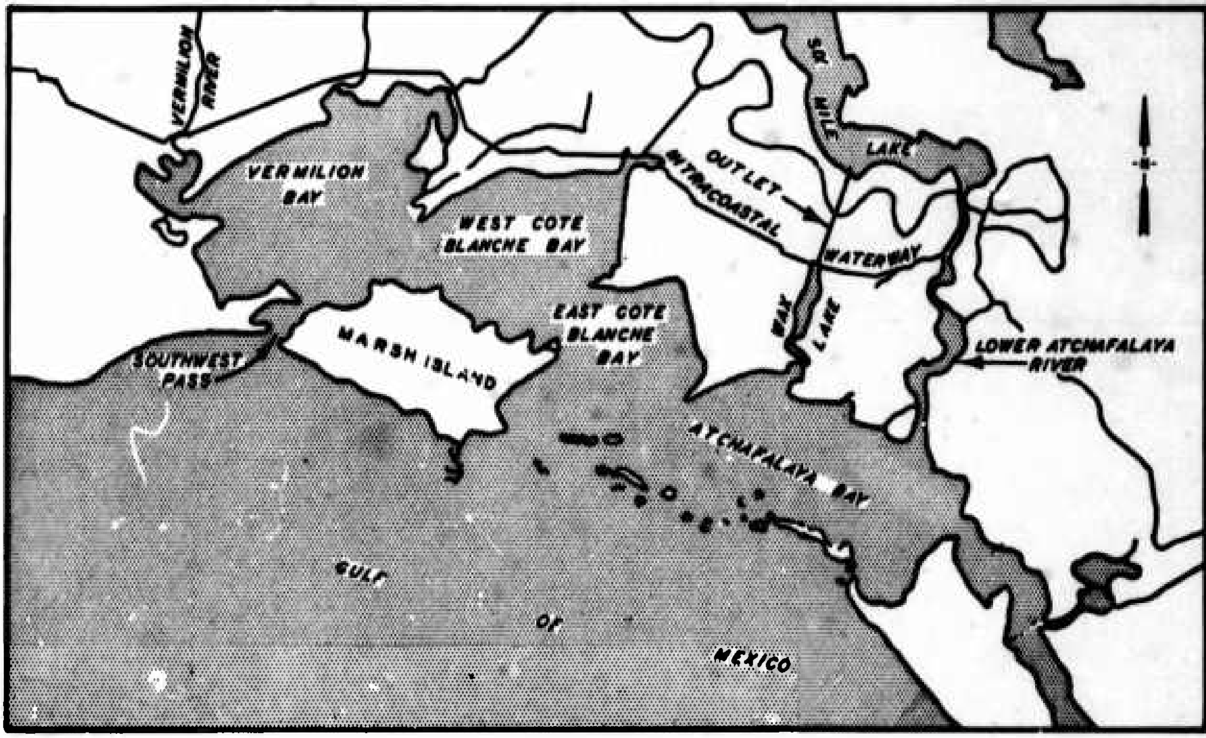

Fig. IX-5. Location of problem area, Vermilion Bay, Louisiana

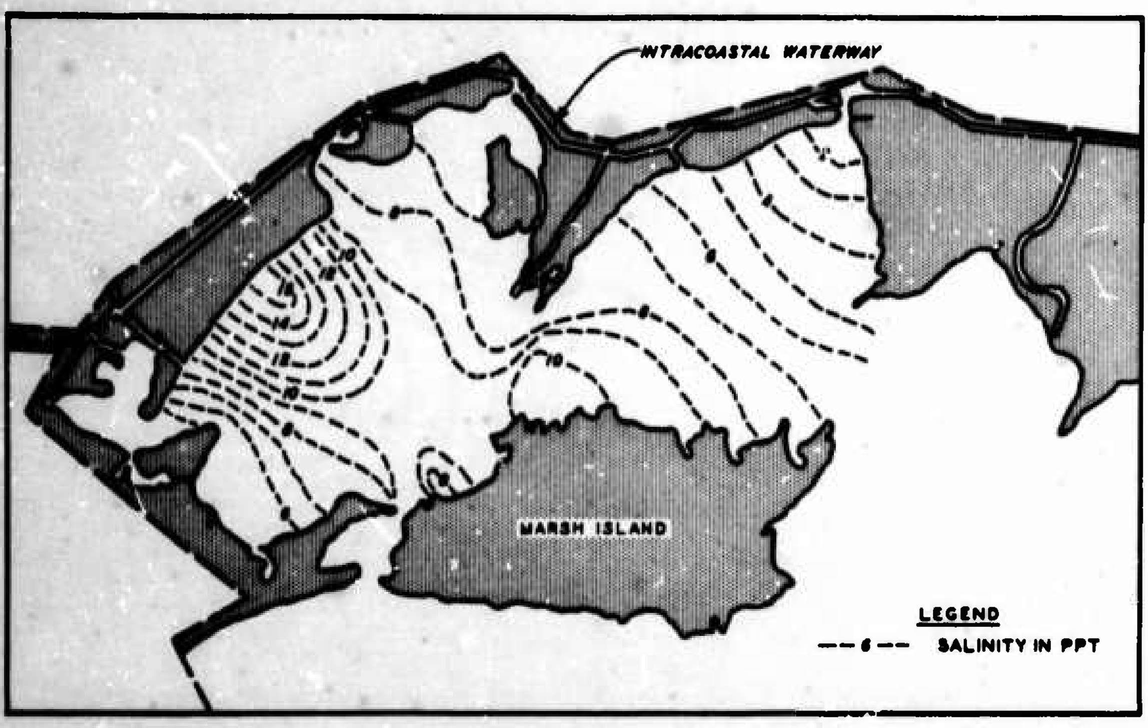

Fig. IX-6. High salinity survey (1954). Southwest Pass open

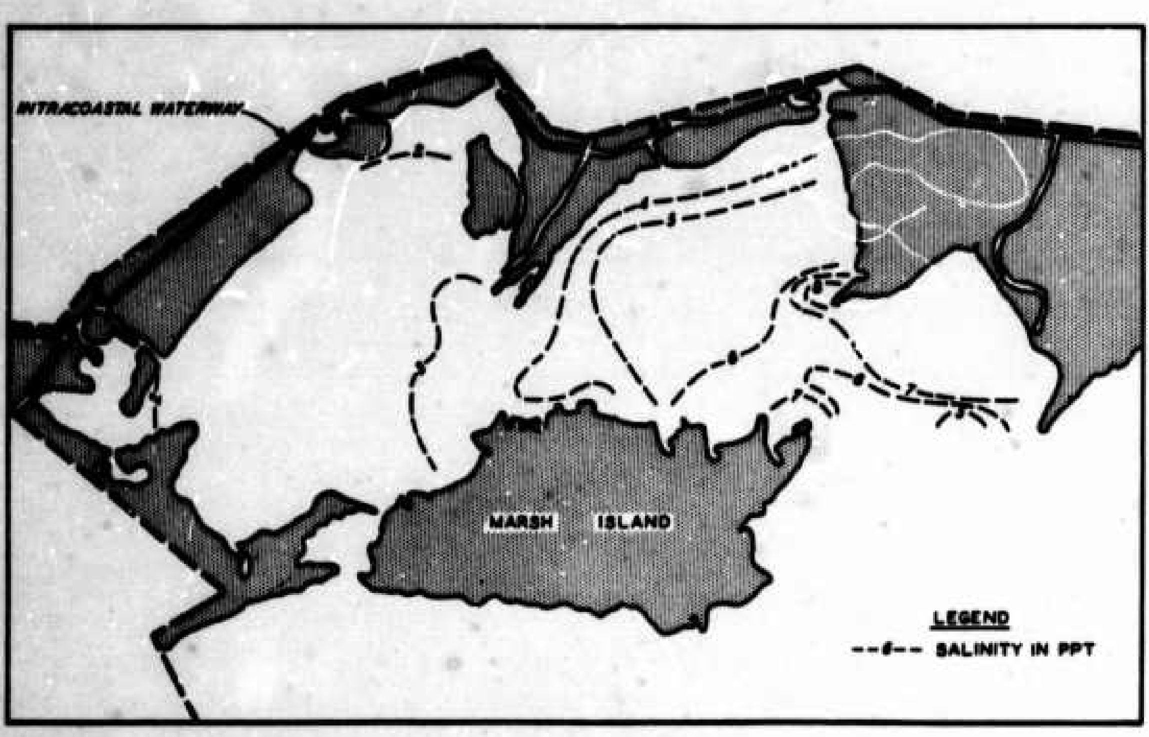

Fig. IX-7. High salinity survey (1954). Southwest Pass closed

1954, which was selected as representing a year of low inflow, and the second made use of inflow data for the water year 1955, which was selected as representing a year of normal inflow. For both conditions, the model was first operated with Southwest Pass open to establish the salinity regimen for existing conditions, and then the pass was closed and the model test was repeated to establish the regimen following closure of the pass.

IX-20. Fig. IX-6 shows the salinity distribution of the bay system at the time of peak salinities for the low inflow year with Southwest Pass open, and fig. IX-7 shows the salinity distribution at the time of peak salinity under similar inflow conditions with the pass closed. Examination of the data presented in these two figures shows that closure of Southwest Pass reduced the maximum salinity along the west side of Vermilion Bay and near the mouth of the Vermilion River from about 15.0 parts per thousand to about 2.0 parts per thousand, or a reduction in maximum salinity of almost 90 percent. Furthermore, while not shown in the figures, the time of maximum salinity at the location was delayed from September to January, which would cause the time of occurrence of maximum salinity to be delayed until well after the end of the irrigation season instead of occurring within the irrigation season. As a result, the salinity reduction afforded by the plan in the critical irrigation season was greater than 90 percent. While this plan has not yet been constructed in nature, the model tests have demonstrated the benefits that would accrue to irrigation interests. This information is vital to long-range planning for the area involved.

Diffusion and Flushing of Wastes

IX-21. The Delaware River model was the first estuary model at WES which was used for studies of the diffusion and flushing of wastes. The limits of this model are shown in fig. IX-8. It reproduces to linear scales of 1:1000 horizontally and 1:100 vertically all of Delaware Bay and the Delaware River to the head of tide at Trenton, New Jersey. The model was carefully adjusted so that tides were reproduced almost exactly to scale throughout its entire length, tidal currents were reproduced accurately in both plan and vertical, and the salinity distribution in both plan and vertical was in good agreement with the prototype for the entire normal range of freshwater discharges into the system. The excellent reproduction by the model of the prototype salinity regimen showed that the mass mixing forces in the model were in good agreement with those of nature, and for that reason it could be assumed that the mass diffusion and flushing patterns of those wastes which would be mixed readily with the waters of the estuary would also be reproduced with good accuracy.

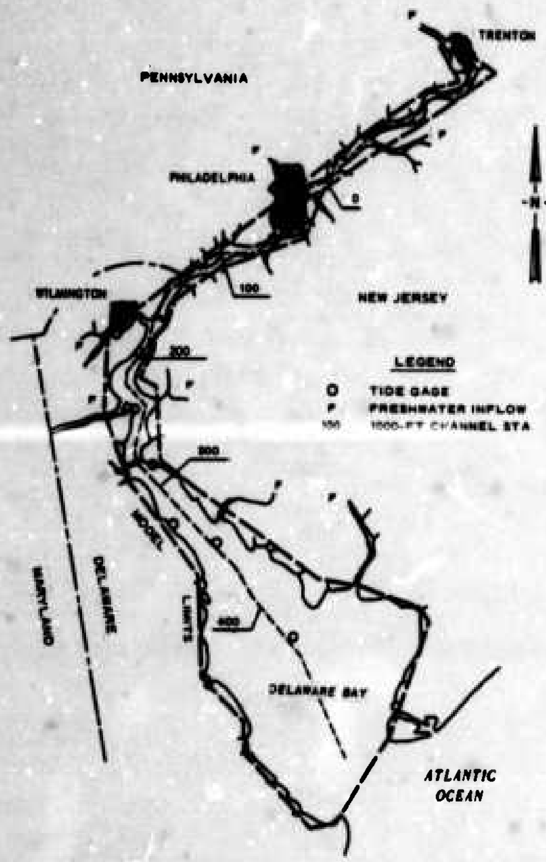

Fig. IX-8. Model limits of Delaware River model

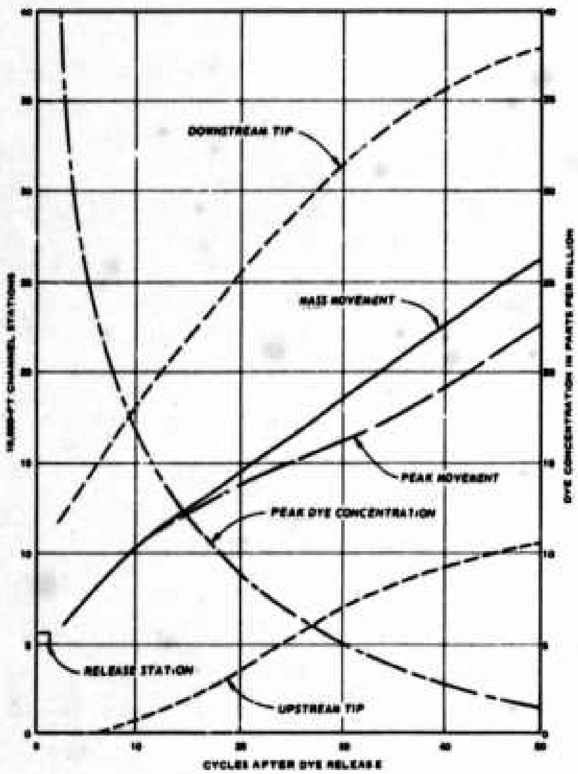

Fig. IX-9. Results of a typical contamination test

IX-22. Some of the earlier studies conducted in this model involved the instantaneous release of relatively large quantities of a dye tracer at various locations in the model, under various conditions of tide and freshwater discharge, followed by continuous sampling operations for periods of 50 or more tidal cycles to determine the rates of diffusion and flushing of the dye tracer. Fig. IX-9 shows the results of a test in which all of the water contained in a 2-ft length of the model channel (2000 ft in prototype), centered at station 50+000, was dyed with methylene blue chloride to an initial concentration of 50 parts per million. Sampling was carried out for 50 tidal cycles after release of the dye. The data in fig. IX-9 show the upstream and downstream limits of the contaminant, the location of the center of mass of the contaminated area, the location of the peak concentration, and the value of the peak concentration, all as a function of time after release.

IX-23. The most significant developments during this test were: (a) immediately after release, the dye spread upstream as well as downstream from the release location, although the rate of spread in the downstream direction exceeded that in the upstream direction; (b) after about 5 tidal cycles following release, the effects of the freshwater discharge prevented further upstream diffusion of the dye, and from this time on the entire contaminated area moved downstream; (c) the center of mass of the contaminant, as well as the peak concentration, moved progressively seaward throughout the test, but the center of mass moved more rapidly than the peak concentration; and (d) the peak concentration was reduced rapidly during the early stages of the test, and more slowly during the later stages, e.g. the initial concentration of 50 parts per million was reduced to 17 parts per million by tidal cycle 10, and to 2 parts per million by tidal cycle 50. A great many dye-diffusion studies have been carried out in the Delaware River and other estuary models, some employing slug releases of the type just described and others employing the continuous releases. The results of these tests have been extremely valuable in defining the diffusion and flushing characteristics of the various estuaries involved, and especially in the planning of waste treatment facilities, waste disposal outfalls, and other works designed to prevent or reduce water pollution in estuaries.

Hydraulic Design of Channels and Disposal of Dredge Spoil

IX-24. A recently completed model study of Matagorda Bay, Texas, is a good example of use of a model of a tidal body for the design of a deep-draft navigation channel, as well as for supplemental studies to determine the optimum locations for dredge spoil disposal during initial excavation and subsequent maintenance dredging of the channel. Matagorda Bay is located on the Texas coast between Galveston and Corpus Christi, and until a 36-ft-deep navigation channel to Point Comfort was recently authorized by Congress, navigation in the bay was limited to a 12-ft-deep channel for barge tows and small craft. The deep-draft project for Matagorda Bay is thus unique in that, instead of the channel being gradually deepened a few feet at a time as has been the case in most estuarine navigation projects, the channel is being dredged in one operation from -12 ft or less to a project depth of -36 ft.

IX-25. The Matagorda Bay model was of the fixed-bed type. It reproduced to linear scales of 1:1000 horizontally and 1:100 vertically the prototype area shown in fig. IX-10. Tides and tidal currents were reproduced by a primary tide generator located in the Gulf of Mexico portion of the model and a secondary tide generator located at the model limit of the channels which connect Matagorda Bay and Espiritu Santo Bay to the west. This secondary generator reproduced the discharge exchange between the two bays which results from tidal action in the bays proper. The model was operated with both salt water and fresh water so that density effects on current velocity distribution in the deep channels would be reproduced, and the effects of the deep-draft project on the salinity regimen of the bay system could be determined.

IX-26. The first, and probably the most important, question that had to be resolved in the Matagorda Bay study was concerned with selection of the optimum route for the entrance channel from the Gulf of Mexico into the bay. Some were of the opinion that the existing Pass Cavallo should be improved by dredging and jetty construction to serve as the entrance channel, and this route is designated Route A in fig. IX-10. Others were of the opinion that a completely new entrance across Matagorda Peninsula should be developed as the entrance, and two such possible routes, designated Routes B and C, are also shown in fig. IX-10. The results of model tests showed that an entrance channel route across the peninsula would be much more desirable than one through Pass Cavallo, from both the standpoints of jetty requirements and flow conditions as they would affect navigation. The tests also showed that flow conditions in an entrance route in the vicinity of Route C would be superior to those in the vicinity of Route B; therefore, the Route C location was selected as the optimum route. In the final design of the entrance, shown in fig. IX-11, the approach channel in the Gulf of Mexico was rotated about 8 deg to the west so as to make this channel normal to the shore, and two 5800-ft-long, rubble-mound jetties, extending from Matagorda Peninsula to the -24-ft-depth contour in the Gulf of Mexico, were incorporated to protect the entrance from wave action and to impound the littoral drift and thus minimize shoaling of the entrance. Two spoil dikes, each about 1000 ft long, extending north along the east and west sides of the channel from the bay shore of

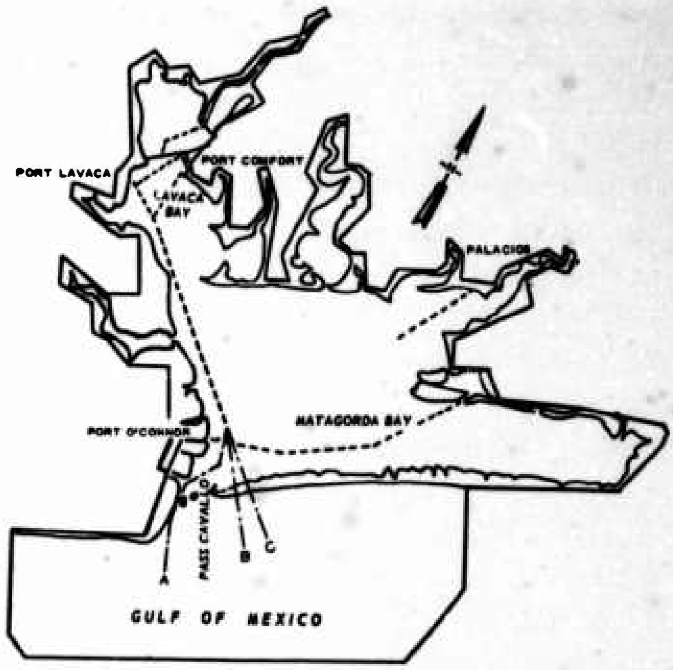

Fig. IX-10. Limits of Matagorda Bay model

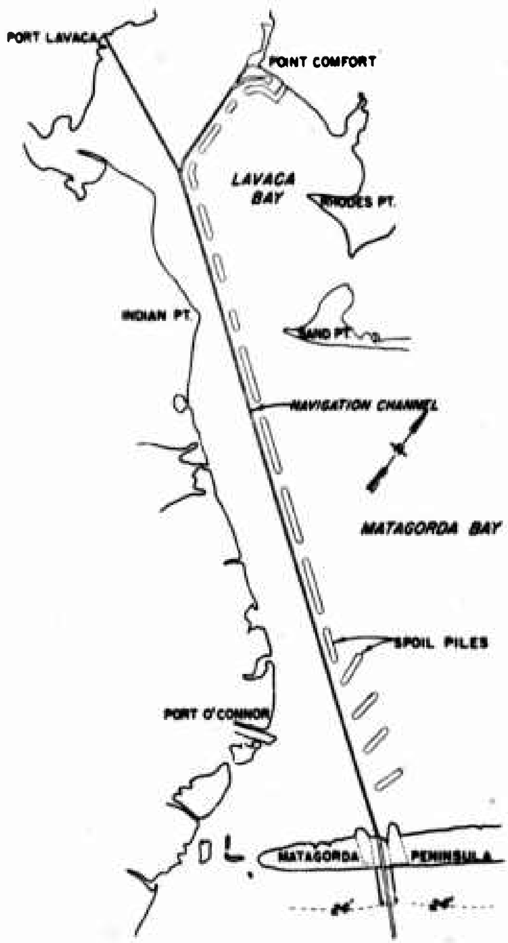

Fig. IX-11. Elements of final plan, Matagorda Bay project

Matagorda Peninsula, were incorporated to improve flow patterns in the navigation channel on the bay side of the peninsula.

IX-27. Because of the large quantity of material that will be dredged in the initial excavation of the channel, and the fact that the dredge spoil cannot be distributed widely in the bay because of redistribution of the material to the channel and possible adverse effects on marine life, the question of proper selection of dredge spoil disposal areas was a highly important one. After numerous tests, it was determined that a chain of spoil banks, generally following the east side of the navigation channel and with substantial openings between adjacent banks as shown in fig. IX-11, would neither result in adverse effects on circulation patterns nor cause undesirable crosscurrents in the navigation channel. In the area between Matagorda Peninsula and a point about opposite Port O'Conner, it was found necessary to orient the spoil banks so that the long axes were parallel to the predominant current directions, thus increasing the widths of openings between adjacent spoil banks to prevent further restriction of the entrance to the bay and the production of crosscurrent velocities and patterns which might be detrimental to navigation. The model tests also indicated that the northerly portion of the west side of the dredge cut through Matagorda Peninsula will probably have to be revetted to prevent excessive erosion by ebb currents in the cut. This information will make it possible to stockpile the revetment materials in strategic locations well in advance for immediate use if the erosion takes place.

Prevention of Tidal Flooding

IX-28. The comprehensive model of Narragansett Bay, the limits of which are shown in fig. IX-12, is an example of use of an estuary model for studies of plans for protection against hurricane surges. Narragansett Bay is located on the New England coast in the states of Rhode Island and Massachusetts. This area has been visited by several tropical hurricanes with disastrous results. For example, the great hurricane of September 1938 crossed the New England coast just west of Narragansett Bay, which route placed the bay in the right front quadrant of the storm, or in the area of maximum winds. As a result, the maximum water level at Providence, Rhode Island, attained a height of about 16 ft above msl or about 13 ft above the predicted height of the tide for that date. Downtown Providence was inundated to depths of 6 to 8 ft, as were other towns and communities throughout the bay system. About 500 lives were lost in the New England area during this storm, and property damage in the Narragansett Bay complex was in the order of $100,000,000.

IX-29. The comprehensive model of Narragansett Bay was of the fixed-bed

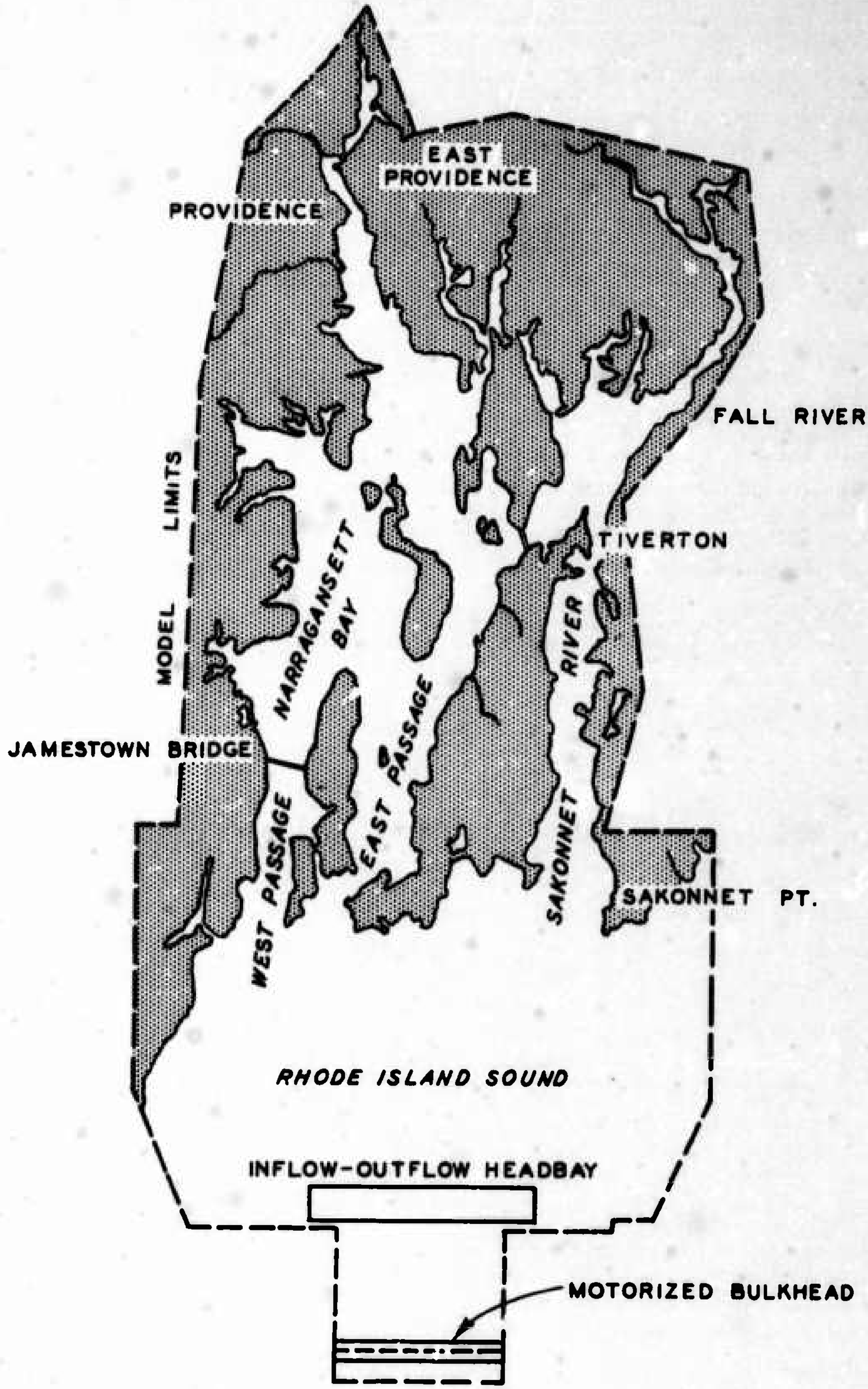

Fig. IX-12. Limits of Narragansett Bay model type and was constructed to linear scales of 1:1000 horizontally and 1:100 vertically. Tides and tidal currents were reproduced by a conventional tide generator, located in the Rhode Island Sound portion of the model. Hurricane surges were simulated by means of a special surge generator, which consisted of a motorized, movable bulkhead, located in a basin adjacent to and connected with the Rhode Island Sound portion of the model. In operation, forward motion of the bulkhead was programmed to displace water from the basin into the model at a rate necessary to reproduce the rising phase of any particular surge, following which backward motion was programmed to allow water to flow from the model into the basin at a rate necessary to reproduce the falling phase of the surge. In this manner, the time-elevation characteristics of any observed or computed surge at the coastline could be reproduced to time and elevation scales, and use of separate generators for normal tides and hurricane surges made it possible to time-phase the two in any desired manner.

IX-30. The absolute elevations attained by a hurricane surge in Narragansett Bay are the products of two primary forces. Firstly, the surge generated in the ocean, through a combination of barometric effects and the drag of hurricane winds blowing over the offshore water surface, moves into and through the bay under the influence of gravity, much as a normal tide moves into and through the bay system. Secondly, the hurricane winds blowing over the bay proper produce local wind-setup effects which tend to increase water levels at the landward end of the bay and to depress water levels at the seaward end. Since it was considered impractical to equip the model to reproduce wind setup because of the large cost involved and other considerations, the model was designed to reproduce the gravitational component of the surge only. Observations made in the model were then adjusted analytically for wind setup, so the surge profiles used for design purposes were derived through a combination of hydraulic model observations and subsequent analytical adjustments.

IX-31. Tests were made of a large number of gated and ungated barriers at various locations throughout the bay, ranging in scope from barriers for protection of the city of Providence alone to lower bay barriers which would provide protection to the entire bay complex. On the basis of these tests, a plan for the protection of Providence, known as the Fox Pt. Barrier, was developed. This plan was subsequently authorized by Congress, and construction is about 50 percent complete. A lower bay barrier plan for protection of the remaining portions of the bay system has been developed through exhaustive tests, and if a few remaining minor problems can be resolved to the satisfaction of all concerned, there is a good chance that this barrier plan will be constructed in the future. In addition to demonstrating how normal tides and hurricane surges would be affected by the various plans under consideration, the comprehensive model of Narragansett Bay was also used to determine how the various barriers would affect current velocities throughout the bay, the distribution of salinities in both plan and vertical, and the diffusion and flushing of pollutants discharged into the bay.

Concluding Remarks

IX-32. As demonstrated by the foregoing specific examples, hydraulic models of estuaries are extremely useful tools in studies leading to a more complete understanding of estuarine phenomena, as well as for predicting the effects of proposed physical and other changes in estuaries on the hydraulic, salinity, shoaling, and flushing characteristics of these complex systems. However, the conduct of such model studies requires considerable experience and judgement, and for this reason estuary models should be entrusted to those who have had much experience in the field and are thoroughly familiar with the uses and limitations of the models involved.

CHAPTER X
DESIGN OF CHANNELS FOR NAVIGATION

by

J. B. McAleer, C. F. Wicker, and J. R. Johnston

Introduction

X-1. The only purpose of a navigation channel is to serve navigation. Therefore, the objective in channel design is to provide that improvement which will meet the needs of the traffic expected to use it. The high cost of construction and maintenance makes it imperative that the waterways engineer develop the design which will obtain the required facilities at the least annual cost. It is the purpose of this chapter to inform the user of the various factors to be considered in the layout of navigation channels. The engineer will be required to exercise a high degree of judgment based on specific information obtained from navigation interests, ship operators and pilots, as well as knowledge of navigation practices and other waterways in the area.

X-2. A ship forfeits open-water maneuverability when it enters a channel. The navigator must be ever alert to the restrictions imposed on his ship. Maneuverability is affected by configuration of the waterway, alignment and dimensions of channel, depth under keel, tidal fluctuations, currents, wave and meteorological conditions, buoyancy, steerage, and interference from other traffic. These problems have always confronted ship captains and pilots but they have been magnified by the trend toward larger and faster ships. Furthermore, large numbers of small craft, both power and sail, increasingly congest our waterways.

X-3. Increasing costs have compelled commercial shippers to exploit improvements in ship design and operational techniques to the maximum. The ship owner's objective is to carry his cargo at the least possible cost per ton-mile, unload, and get under way on the next trip. The largest World War II cargo ships and tankers, which ranged up to 16,500 DWT, 523 ft in length, and 30-ft draft are being replaced by super ships of over 100,000 DWT, with lengths approaching 1000 ft, drafts close to 50 ft, and 130-ft beams. The SS MANHATTAN delivered in 1962 is 106,568 DWT, has an overall length of 940 ft, molded beam of 132 ft, loaded draft of more than 49 ft, and a sea speed of nearly 18 knots. It is obvious that as vessel size increases, at some point the economics of ship operation and of channel construction and maintenance must conflict. However, this chapter deals only with the technical aspects of safe and economic design of channels.

X-4. The design of new or enlarged navigation channels in estuaries must give consideration to the effect of the tidal forces on the channels as well as any effect of the channels on the tidal regimen. They may cause major changes in tidal currents, sediment movements, shoreline configuration, salinity intrusions, and mixing processes. These considerations are described in earlier chapters.

Channel Depth

X-5. Experience has established that adequate depth is the first requirement of safe navigation in a waterway. Channel depths substantially greater than the loaded static drafts of the vessels using the waterway are required in order to ensure safe and economic navigation. Therefore, in the design of a channel, the minimum depth would be first considered, and then the width and other requirements.

X-6. Common practice on the east and gulf coasts of the United States is to establish depths at mean low water (mlw) as the design depths, and on the west coast to establish the depths at mean lower low water (mllw). In cases where the traffic density is great and tides frequently occur below the datum, a plane below mlw or mllw is frequently selected. On the other hand, where traffic density is low, the design depth may be set for a higher tide level, such as half tide level. The basis for selection will be an economic analysis of the cost of vessel delays, operation, and light-loading balanced against dredging costs. Each reach of the vessel's entire round trip on a waterway needs to be considered, as a ship can frequently carry the flood tide throughout most of its journey upstream, but if the waterway is long, the vessel may have to anchor during a downstream passage and await the rising tide.

X-7. Data on local operating practices and vessel characteristics are essential elements in the design of economical channels to meet the requirements of practical ship operation. This applies to the determination of depth and other characteristics as well.

Loaded draft of design vessel

X-8. Table X-1 gives the loaded draft and other characteristics of typical existing ships that move in the U. S. trade. The channel design will take into consideration not only the requirements of present-day vessels that will use the waterways but also the trend in size of vessels. Data on general trends may be seen in General Cargo Vessels - Trends and Characteristics,[1*] Study of Trends

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* Raised numerals refer to similarly numbered items in Selected Bibliography at the end of this chapter.

in Petroleum Supply Requirements and Tanker Fleets and Characteristics,[2] and Trends in Dry Bulk Carriers.[3] "Loaded draft" usually refers to the draft amidships of a vessel at rest when loaded to the summer load line. Actual loadings may be less than this, and they may be such as to cause a greater draft aft than forward, or occasionally vice versa.

Table X-1

Characteristics of Representative Vessels

Type	Deadweight	Length Overall		Beam		Loaded Draft*		Speed
	tons	ft	in.	ft	in.	ft	in.	knots
Tankers								
T-2 Jumbo	20,000	572	--	75	--	30	2	14.5
	23,000 to 25,000	577	--	78	9	32	6	16.0
	30,000 to 32,000	654	--	86	--	34	2	17.0
	38,000 to 40,000	715	--	93	2	36	7	17.0
	48,000 to 50,000	733	--	102	--	38	9	17.0
	60,000	810	--	104	8	41	8	17.0
	67,000	860	--	104	--	43	[illegible]	17.0
	85,000	845	--	125	--	46	3	14.5
	106,500	940.6	--	132	--	49	--	17.5
Bulk Carriers								
EC2-S-AWI (Liberty Collier)	11,000	441	6	56	11	28	6	11.0
Bauxite, alumina	13,700	518	--	66	--	27	9	14.5
Bauxite, alumina	17,300	534	--	70	--	30	6	16.0
Ore, petroleum	20,500	633	1	71	10	32	10	16.8
Ore, grain, phosphate	24,000	582	11	78	4	34	4	16.0
Ore carrier (bucket unloader)	24,000	605	--	75	--	33	7	16.0
Ore carrier (self unloader)	26,000	620 to 625	--	80	--	34	2	16.0
Iron ore carrier	32,000	662	--	87	--	33	11	16.0
Ore carrier	34,200	680	--	88	4	34	6	16.0
Ore carrier	46,500	744	--	100	6	37	2	16.5
Ore carrier	60,000	794	--	116	--	38	8	14.0
Dry Cargo								
N3	2,900	258	9	42	1	17	11	11.0
R1-M (Reefer)	4,373	338	9	50	--	21	--	11.0
C1-M	5,100	338	9	50	--	21	--	11.0
C-1-B	9,100	417	9	60	--	27	6	14.0
C2	9,200	459	3	63	--	25	9	15.5
C3	12,300	492	--	69	6	28	6	16.5
C4 Mariner	12,900	560	--	76	--	29	11	20.0
C4	14,900	522	11	71	6	32	10	16.5

* Draft amidships when loaded to the vessel's saltwater summer load line.

Freshwater draft

X-9. In passing from sea water having a density of 1.026 (64 lb/cu ft) into fresh water with density of 0.9991 (62.4 lb/cu ft), a vessel's displacement must increase approximately 3 percent. The vessel will sink from 2 to 3 percent of its draft, depending upon the hull design. A vessel with a 35-ft saltwater draft, for example, would have a freshwater draft of about 36 ft with intermediate drafts in brackish waters.

Sinkage due to squat

X-10. A ship in motion will apparently sink or "squat" an amount depending on (a) the speed of the vessel through the water, (b) the distance between the keel and the bottom, (c) the trim of the vessel, (d) the cross-sectional area of the channel, and whether the channel is located in a wide or narrow waterway, (e) whether the vessel is passing or overtaking another vessel, (f) the location of the vessel relative to the center line of the channel, and (g) the characteristics of the ship itself. In fact, the ship does not sink relative to the water, but instead there is a lowering of the water surface due to the passage of the ship, and this of course causes the ship to be closer to the bottom while in motion over a given location than it would be at rest over the same location.

X-11. When a vessel is in motion in still water, the water ahead of the vessel is moved forward, outward, and downward. A short distance aft of the bow, the water moves aft, outward, and downward to make way for the body of the ship. These changes in velocity cause changes in the elevation of the water surface, in accordance with Bernoulli's theorem. The swiftest flow occurs amidships, and the elevation of the water surface in the vicinity of amidships is at its lowest. In the open ocean, where depths are great and for all practical considerations there are no limits in the x and y directions, the currents generated by the moving vessel are much lower than in the case of shallow and restricted waterways. Consequently, the change in water surface elevation in the open ocean due to the passage of a ship will be relatively small while the change in a shallow and narrow waterway will be relatively large. Thus, for a given vessel moving at a given speed (within limits) in a restricted waterway (i.e. one where the navigation channel occupies most or all of the cross section of the waterway), the squat will be greater than would be experienced in the open ocean. Additional depth must be provided in such channels for safe navigation.

X-12. After making certain simplifying assumptions, the Bernoulli theorem is applicable for analysis of the phenomenon:

$$[(v_1)^2/2g] + h_1 = [(v_2)^2/2g] + h_2 \qquad \text{(X-1)}$$

Combining this equation with the equation of continuity

$$v_1 w h_1 = v_2 (w h_2 - A) \quad \text{(X-2)}$$

and introducing certain dimensionless parameters, it can be shown that:

$$F = \left(\frac{2d\,(1 - d - s)^2}{1 - (1 - d - s)^2} \right)^{1/2} \quad \text{(X-3)}$$

where

v_1 = the speed of the ship relative to the water

h_1 = the undisturbed mean depth of the water

v_2 = the velocity of the currents in the cross section occupied by the vessel, measured relative to the vessel

h_2 = the depth of water in the cross section occupied by the vessel

w = the width of the waterway

A = the wetted cross-sectional area of the ship at its midsection

F = the Froude Number

d = the dimensionless "squat" = z/h_1 where $z = h_1 - h_2$

s = the ratio of ship cross section A to channel cross section $h_1 w$

Equation X-3 is essentially that developed by Constantine,[4] but with other notations and presentation, the same equation has been developed by Taylor Model Basin in Report 601[5] and Schijf.[6]

X-13. Fig. X-1 (page X-8), a plot of equation X-3 for a range of values of s, shows clearly that the dimensionless squat parameter d increases at moderate rates as the Froude number increases until some apparently critical value of F is attained; thereafter, the squat increases with great rapidity. It is seen that each curve becomes asymptotic at some value of F; apparently it is not possible for the speed of the vessel to exceed a certain value, depending on the depth and the relation between the cross-sectional area of the vessel and that of the waterway, regardless of the propulsion effort exerted. According to Schijf,[6] increases in the power output of the engines of the vessel when it is already moving at or near the limiting speed results in great turbulence behind the vessel without notable increase in speed.

X-14. The values of v_1 in the Froude number F are relative to the vessel; in still water, v_1 is the speed of the vessel over the bottom of the channel. If there is a current in the waterway flowing in a direction opposite to that of the travel of the vessel, the speed of that current will be added to the speed of the vessel over the bottom of the waterway; if the current is in the same direction as the travel of the vessel, its speed will be subtracted from that of the vessel. The depth h_1 in both F and d is the mean depth of the waterway.

X-15. A search of the literature for squat observations made in the detail necessary for comparison with the results obtained by use of equation X-3 produced 38 examples covering a wide range of conditions. Some of the observations were made in prototype waterways, while others resulted from experimentation with models. Table X-2 gives the particulars concerning the vessels, the waterways involved, and the computed and observed squats.

X-16. Table X-2 shows that equation X-3 yields results that are reasonably close to the observed values over a wide range of waterway dimensions and vessel sizes and speeds. The values compared in entries 9 through 27 and 28 through 33 are representative of a large number of observations made through a range of vessel speeds in the respective models, but it was not considered necessary to include a greater number of values in the tabulation, as the results would have been comparable to the values used. Fig. X-2, a plot of the computed squat figures against the average of the bow and stern squats, shows that some of the computed values are less than the observed values. Although some of these departures may be due to errors in observation or of reduction of vessel speed over the bottom to vessel speed through water, in cases where there is a current, it is likely that many are due to the simplifying assumptions made in developing equation X-3. It is notable that the greatest departures occur in cases where the ratio of vessel to waterway cross sections is very small, as in the observations made in the Delaware.

X-17. A paper[7] submitted to the XXth Congress of the Permanent International Association of Navigation Congresses (PIANC) on behalf of the Royal Dutch Shell Group of Companies reports on investigations made by the Sogreah Laboratory at Grenoble, France, to formulate general rules which could be applied to any required set of conditions. It appeared that a basis of this investigation was the Schijf investigations reported on at the XVIIth and XVIIIth Congresses of PIANC and already referred to herein.[6] The Sogreah Laboratory developed a graphical method for determining squat that is shown in fig. X-3. The data tabulated in table X-2 as to the characteristics of the waterway and of the vessels, including vessel speed relative to water, were utilized in making a new set of computed squat values using the Sogreah graphs (fig. X-3). The results are plotted in fig. X-4.

X-18. A comparison of figs. X-2 and X-4 shows that the Sogreah method produces computed values of squat that are closer to the observed values than does the use of equation X-3 when the squat is less than 2 ft, but that the Sogreah method results in computed values that are not as close to the observed values as those computed by equation X-3 for squats greater than 2 ft. A closer study of the departures of computed values from observed values shows that the Sogreah method is likely to produce values that are close to the observed when the value

Table X-2

Comparison of Computed and Observed Vessel Squats

Entry No.	Vessel		Waterway			s*	Ship Speed knots	Squat		
	Beam	Static Draft ft	Naviga-tion Depth ft	Mean Depth ft	x-section sq ft			Com-puted ft	Observed	
									Bow ft	Stern ft
Panama Canal**										
1	78.3	35.5	39.0	31.0	34,100	0.081	9.9	1.30	2.16	1.85
2	78.3	35.5	39.0	39.0	14,430	0.192	6.1	1.05	1.22	1.26
3	62.5	26.9	39.0	31.0	34,100	0.049	10.0	0.78	1.41	0.77
4	62.5	26.9	39.0	39.0	14,430	0.116	7.0	0.62	1.02	0.48
5	57.1	19.3	39.0	31.0	34,100	0.032	9.3	0.37	0.86	0.72
6	57.1	19.3	39.0	39.0	14,430	0.076	7.0	0.39	0.55	0.18
7	57.1	26.9	40.0	31.9	35,700	0.044	9.6	0.54	1.14	0.83
8	57.1	26.9	40.0	40.0	14,800	0.104	6.4	0.52	0.81	0.50
Panama Canal†										
9	113.0	32.2	59.5	55.2	45,200	0.081	14.0	2.87	3.00	2.74
10	108.0	34.6	60.0	55.0	39,600	0.092	12.0	2.03	2.28	2.08
11	108.0	34.6	60.0	55.0	39,600	0.092	14.0	3.58	3.65	3.37
12	113.0	32.2	60.0	54.2	33,600	0.108	12.0	2.54	2.44	2.44
13	113.0	32.2	60.0	54.2	33,600	0.108	14.0	4.32	4.05	4.05
14	113.0	32.2	45.0	42.5	33,600	0.108	11.5	2.55	2.59	2.59
15	113.0	32.2	45.0	42.5	33,600	0.108	12.0	3.39	3.07	3.07
16	113.0	32.2	45.0	42.5	33,600	0.108	12.5	3.49	3.90	3.90
17	113.0	32.2	81.6	67.0	31,000	0.118	12.1	2.48	2.59	2.25
18	113.0	32.2	81.6	67.0	31,000	0.118	13.4	3.35	3.53	3.04
19	113.0	32.2	45.0	41.5	24,500	0.148	9.4	2.07	2.21	2.21
20	113.0	32.2	45.0	41.5	24,500	0.148	11.4	4.97	4.24	4.05
21	113.0	32.2	61.4	52.5	22,200	0.164	5.7	0.73	0.83	0.64
22	113.0	32.2	61.4	52.5	22,200	0.164	10.7	3.41	2.70	2.89
23	113.0	32.2	61.4	52.5	22,200	0.164	11.7	4.20	3.45	3.83
24	100.0	32.1	45.0	40.9	19,500	0.165	8.0	1.56	2.19	1.43
25	100.0	32.1	45.0	40.9	19,500	0.165	9.0	2.12	3.06	1.93
26	113.0	32.2	45.0	39.9	15,500	0.235	8.1	3.07	2.96	2.78
27	113.0	32.2	45.0	39.9	15,500	0.235	9.1	4.80	4.05	4.24
TEMB††										
28	62.0	28.5	40.0	40.0	38,000	0.046	10.0	0.64	1.45	1.20
29	62.0	28.5	31.0	31.0	29,500	0.060	6.0	0.22	0.63	0.47
30	56.9	27.7	31.0	31.0	28,200	0.056	5.5	0.15	0.60	0.27
31	56.9	27.7	40.0	40.0	36,400	0.043	9.0	0.40	1.21	0.59
32	68.0	30.2	33.0	33.0	28,200	0.072	7.0	0.40	0.88	0.87
33	68.0	30.2	40.0	40.0	34,200	0.060	6.0	0.20	0.45	0.38
Delaware‡										
34	64.0	19.5	45.3	24.8	200,000	0.006	12.6	0.30	1.22	2.06
35	58.0	19.4	44.3	23.3	186,000	0.006	12.6	0.28	0.93	1.43
36	58.0	18.5	40.3	16.6	129,000	0.008	11.3	0.44	0.58	1.72
37	58.0	18.4	44.0	22.7	182,000	0.006	12.0	0.13	0.90	1.09
38	58.0	16.4	38.1	15.2	116,000	0.008	11.6	0.64	0.20	1.41

* s = ratio of vessel submerged cross section to waterway cross section.

** Measurements made in prototype canal (1947) as reported in Taylor Model Basin Report No. 601.[5]

† Measurements made in model canals as reported in Taylor Model Basin Report No. 601 (1948).[5]

†† Measurements made in Experimental Model Basin, Naval Gun Factory, Washington, as reported in Taylor Model Basin Report No. 640 (1948).[8]

‡ Measurements made in prototype Delaware River (1936) as reported in Appendix IX, Senate Document 159, 75th Congress, 3d Session.

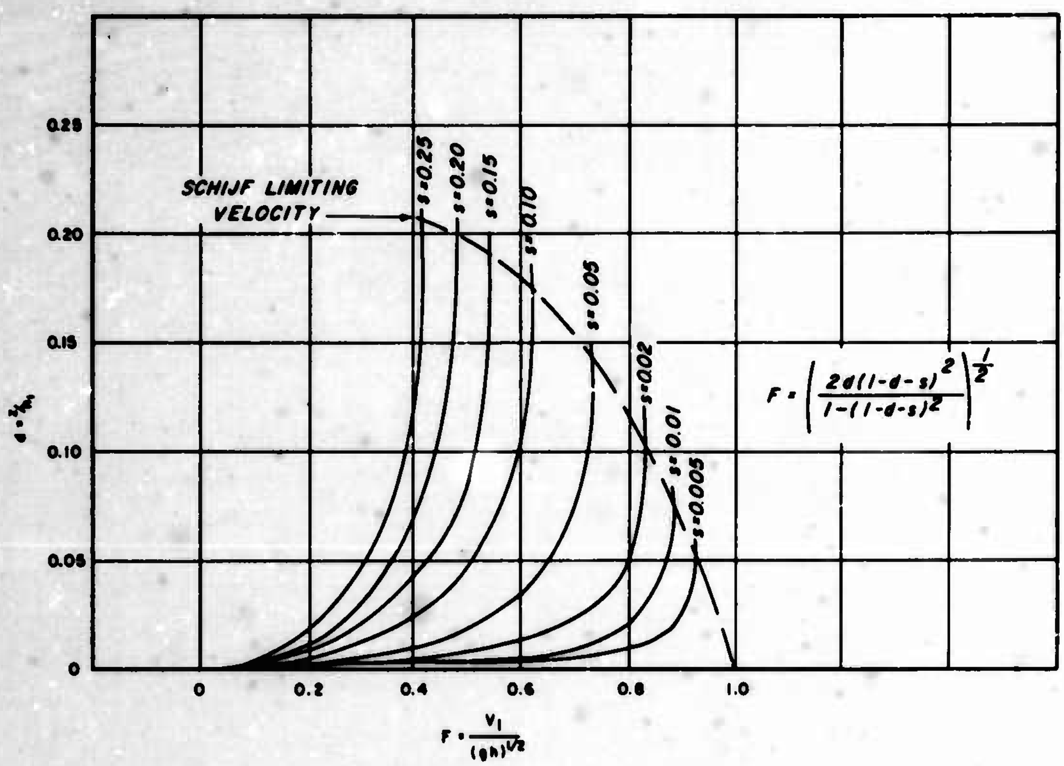


Fig. X-1. Dimensionless squat as a function of the Froude number

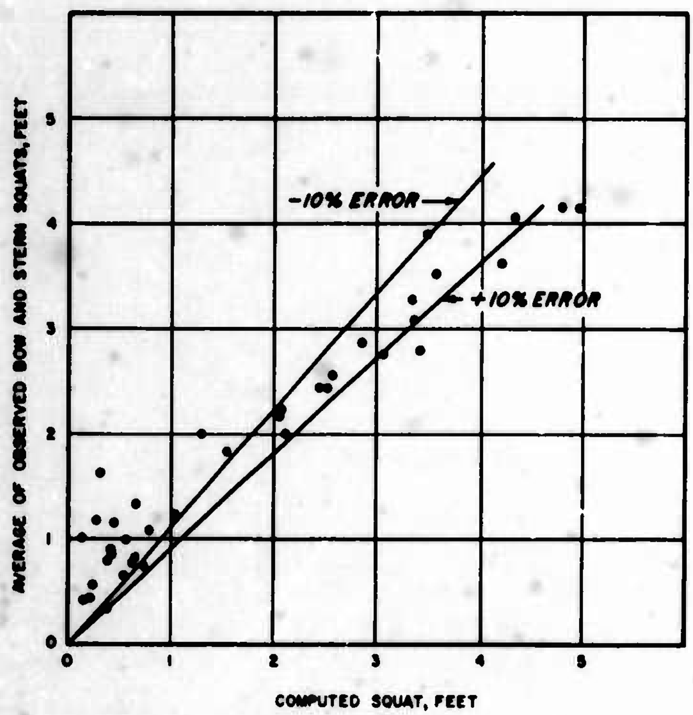


Fig. X-2. Observed versus computed squats (Computations made by use of equation X-3)

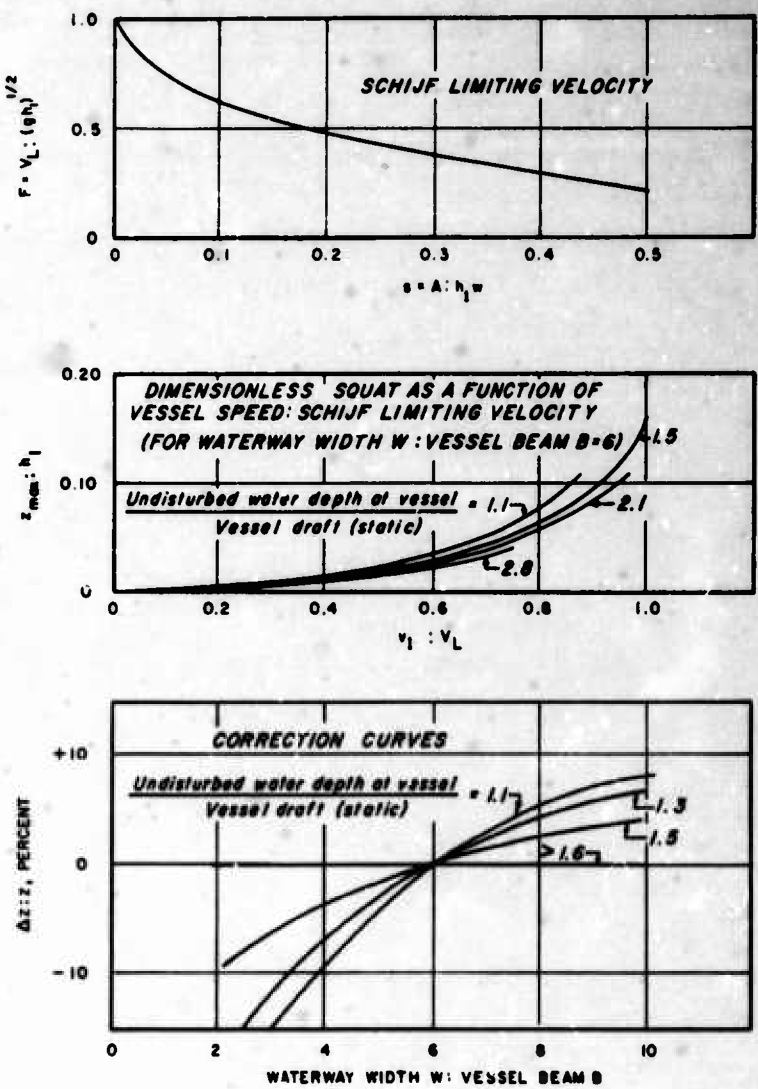


Fig. X-3. Sogreah Laboratory squat curves

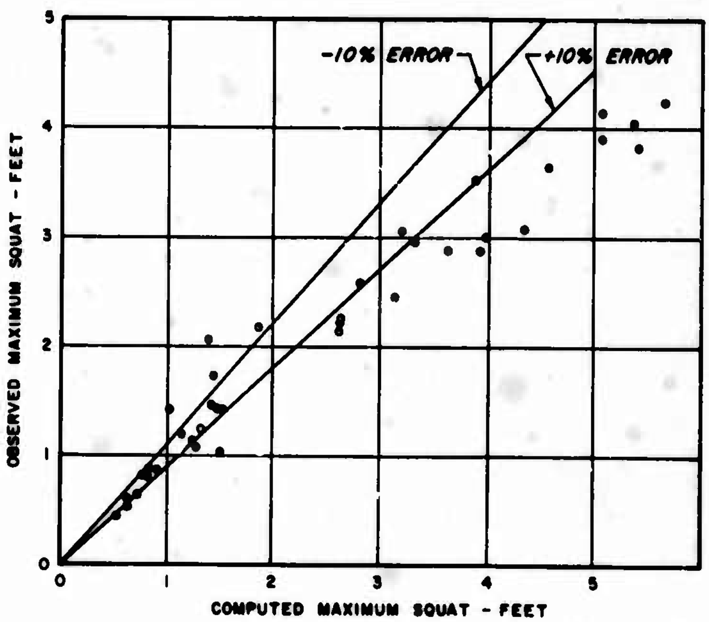


Fig. X-4. Observed versus computed squat (Computations made by Sogreah method) of s is less than 0.080, and that equation X-3 provides better results when s is greater than 0.080. Fig. X-5 is a plot of computed vs observed squats on this basis. It is considered that this check shows that the use of equation X-3 and the Sogreah method in combination provides results that can be relied upon for channel design purposes.

X-19. Equation X-3 may of course be applied directly, but the use of the graphs in fig. X-1 makes its application much easier. Knowing the vessel speed relative to water (v_1 in fps), and the mean depth in the cross section (h_1 in ft), the Froude number F is readily computed. Knowing the static draft of the vessel in water of the density of that in the cross section under consideration, and the beam of the vessel, the vessel's cross-sectional area A is readily computed. Knowing the width of the waterway w and the mean depth h_1, the cross-sectional area of the waterway is computed. The dimensionless figure s, the ratio of the ship cross-sectional area to the cross-sectional area of the waterway, is computed. Entering the graphs in fig. X-1 at the computed Froude number F, follow this ordinate to its intersection with the graph corresponding to the computed s, or to some interpolated value. Read the value d at this intersection, and compute the squat z by multiplying d by h_1.

X-20. Computations by the Sogreah method are made as follows. First, utilize the "Schijf limiting velocity" graph (fig. X-3) by entering at the computed s value (determined as shown previously) and read the Froude number F. Compute the limiting velocity V_L in fps by multiplying F by $(gh_1)^{1/2}$, and use the value thus determined to compute the ratio $v_1:V_L$ where v_1 is the speed of the vessel in fps relative to the water (as in paragraph X-14). Use this figure to enter the graphs in the middle section of fig. X-3 ("Dimensionless Squat as a Function of Vessel Speed and Schijf Limiting Velocity"). Move up to the intersection with the graph corresponding to the ratio of the undisturbed depth of water at the vessel (<u>not</u> the mean depth in the cross section h_1) to the static draft of the vessel, read the dimensionless squat $z_{max}:h_1$, and compute z_{max} by multiplying this ratio by h_1, the <u>mean</u> depth of the waterway (not the depth of water at the ship). The graphs in the middle of fig. X-3 are computed for the ratio waterway width w:vessel beam B = 6. Other ratios may require adjustment of the value of the squat determined as above, and the graphs in the bottom section of the figure may be used for this purpose. Enter these curves at the ratio of waterway width to vessel beam and move up to its intersection with the curve for the ratio of the undisturbed water depth at the vessel to the static draft of the vessel, as computed above. For ratios greater than 1.6, no correction is necessary. Correct z_{max} by the percentage indicated, noting that the correction may be plus or minus, depending on whether the ratio of waterway width to vessel beam is greater or less than 6.

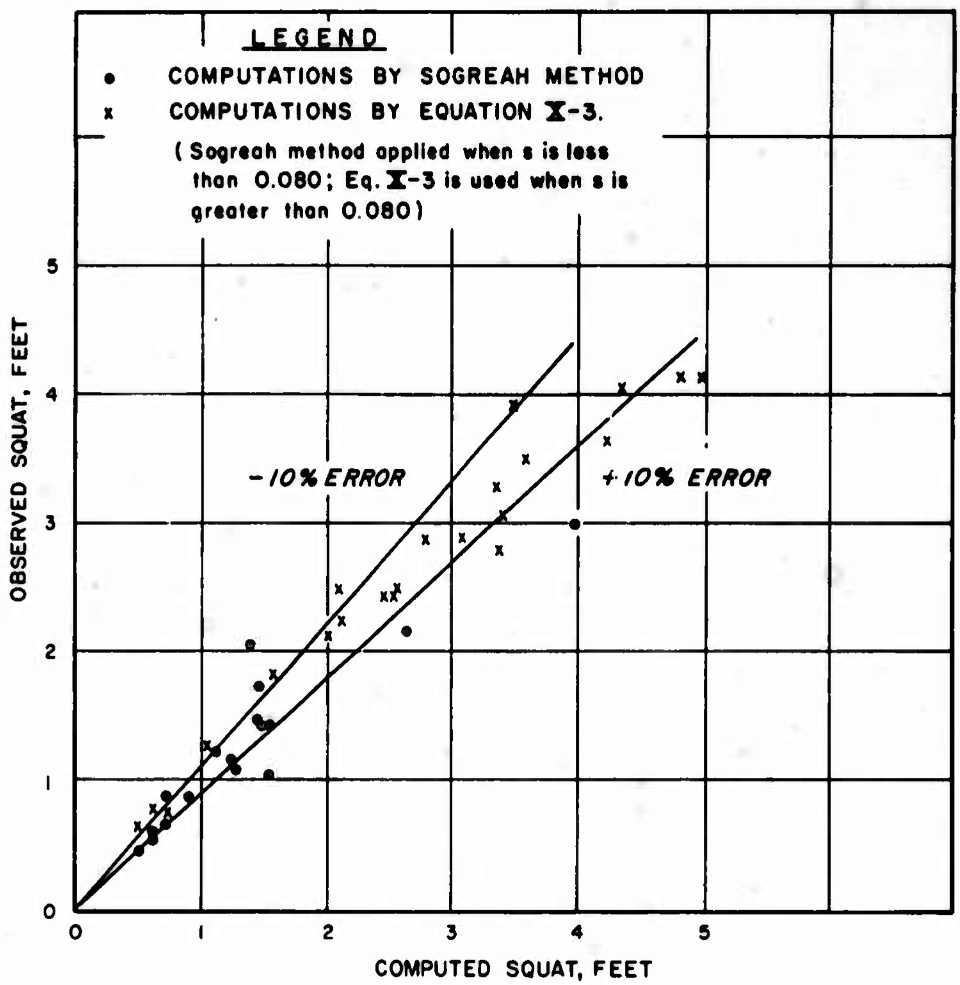


Fig. X-5. Observed versus computed squat

X-21. It was stated in paragraph X-10 that squat is a function of whether the vessel is overtaking or passing another vessel, as well as of a number of other considerations, many of which have already been discussed. It is also a function of the position of the vessel in the channel, i.e. whether on the center line or off to one side. Obviously, when vessels are meeting or passing each other, it is likely that each of them will be off to the side of the center line, and that the squats of each of them may be greater than would be the case if they were alone in channel and at the center line. Although little experimental data could be located to permit formulation of definite findings on this matter, it appears that estimates of the increased squat of vessels passing each other would be made by first determining the increase due to the presence of the other vessel, then adding to the result the increase due to the proximity of the bank of the waterway or channel. It appears reasonable to assume, for example, that the computations for the squats of two identical vessels loaded to the same drafts passing in a restricted waterway of such dimensions that the s value is greater than 0.080 would be based on a deduction from the cross-sectional area of the waterway of the cross-sectional area of the vessel being passed. Thus, disregarding the bank effect for the present, if there are two vessels of the size and draft listed in entry 19 of table X-2 passing in the canal indicated in this entry, the squats of each of them would be computed by decreasing the cross-sectional area of the waterway by the cross-sectional area of one of the vessels, $113 \times 32.1 = 3640$ sq ft, making the cross-sectional area of the waterway $24{,}500 - 3640 = 20{,}860$ sq ft. The effective s then becomes $3640 \div 20{,}860 = 0.175$. The Froude number F for entry to the graphs in fig. X-1 is 0.43, as is the case for the condition tabulated in entry 19 in table X-2, but $s = 0.175$ will be used instead of $s = 0.148$, as in entry 19, and d is seen to be 0.64 making the squat 2.65 ft instead of 2.07 ft. If it is now assumed that each vessel maneuvers itself so that its starboard side is 26 ft from the edge of the canal bottom, the squat will be increased about 50 percent from 2.65 to about 4.0 ft. This is based on experimental data[5] discussed below.

X-22. Graphs and tables are available[5] that permit comparison of squats of a given vessel in canals of various geometry running on the center line and at two or three positions off the center line. The available data sampled for vessel speeds of 5 and 8 knots are shown in table X-3, page X-14; the data contained in reference 5 are for a full range of vessel speeds. Fig. X-6, a plot of all of the data for the 45- by 500-ft canal listed in table X-3, is included in order to illustrate how the squat increases with speed at the several locations of the ship.

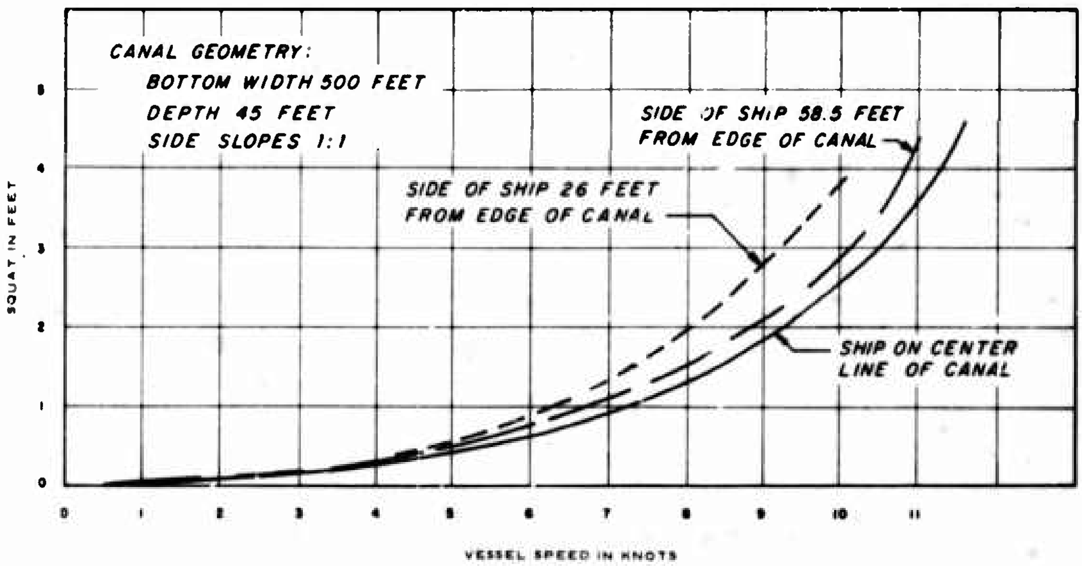


Fig. X-6. Effect of ship's location in canal on squat

Table X-3

Effect of Ship's Location in Channel on Squat

Channel Dimensions ft	Location of Ship	Squat for Ship Speeds Indicated, ft	
		5 knots	8 knots
45 by 300	On center line	0.79	2.82
45 by 300	26.0 ft from edge*	0.86	2.81
60 by 300	On center line	0.55	1.31
60 by 300	48.5 ft from edge	0.55	1.42
60 by 300	26.0 ft from edge	0.59	1.50
45 by 500	On center line	0.40	1.30
45 by 500	58.5 ft from edge	0.52	1.51
45 by 500	26.0 ft from edge	0.50	2.00
60 by 500	On center line	0.30	0.90
60 by 500	26.0 ft from edge	0.38	1.40
45 by 700	On center line	0.38	1.02
45 by 700	26.0 ft from edge	0.48	1.56
60 by 700	On center line	0.23	0.73
60 by 700	26.0 ft from edge	0.34	0.93

* The edge referred to is the edge of the bottom of the canal at the depth shown. The distance is from the side of the ship. The ship involved in these tests had a static draft of 32.25 ft and a beam of 113 ft.

X-23. These data show that the increase in squat over that experienced by the vessel when it is at the center line is small at the lower speeds, but becomes significant for higher speeds. The data also show that squat for a given vessel speed increases as the edge of the channel is approached. The squat that occurs when the vessel is operated 26 ft from the edge of the channel is on the order of 50 percent greater than that occurring when the ship is on the center line of the channel. The percentage increase when the ship is closer to the center line is of course somewhat less.

X-24. Thus, in the example given in paragraph X-21, the total squat of the vessel when it is passing another similar vessel and its side is only 26 ft from the edge of the canal will be 4.0 ft, but the squat of this same vessel when it is alone in the channel and is sailing on the center line is 2.1 ft. As stated previously, there are little experimental data to support this, but in addition to the data in reference 5, there is a statement in the Sogreah report[7] summarizing the results of experiments that showed the squat when two vessels passed each other at speeds of 4.75 knots was double that when each vessel was alone in the channel, while the squat was in the region of 1.5 times the normal value at higher speeds.

It is seen that these data tend to corroborate the findings described previously. It is considered that the additional squat that occurs when vessels pass each other is significant, and that allowance should be made for additional depth when it is likely that the ship for which the channel is designed will pass another ship of similar characteristics.

X-25. Vessel trim is also a factor in the amount of squat experienced, although the literature available for this summary of knowledge pertaining to the matter did not provide definitive results on the effect of uneven trim as compared to even trim. It seems likely that a vessel down at the stern while at rest might have a greater squat at the stern than would be experienced if the static bow and stern drafts were the same, other things being equal, but no quantitative data on this matter are available. For purposes of the design of suitable channel depths, it may be assumed that the computed squat should be applied to the greatest static draft of the vessel, whether it be the bow or the stern, to determine the greatest draft in motion.

Effect of pitching and rolling

X-26. Pitch, roll, and heave (which is the vertical motion of ship's center of gravity) occur under the influence of waves. In open sea conditions, a pitch angle of 2.5 deg in a 1000-ft ship would increase draft forward by about 22 ft. A 5-deg angle of roll for a ship having a beam of 100 ft would increase amidships draft about 4.4 ft. This is not an unusual roll at entrances, even in semiprotected conditions, as a result of waves, wind, and turn angle.

Effect of depth on power requirements

X-27. The factors that produce sinkage also slow down the vessel. If good speed can be maintained in shallow water, it can be done only at the expense of great power and fuel consumption as compared with deepwater operation. A rough approximation of the increase in resistance or power with depth at speeds well below critical (Schijf limiting velocity) is given by the formula

$$\text{Percent increase in resistance} = \frac{50D}{D_w}$$

where

D_w is the depth of water and D is the draft of the ship.

X-28. In 40-ft water depth a vessel having a draft of 30 ft would require about 37.5 percent additional power over the same speed in deep water. Fig. X-7 (from reference 8) shows increased shaft horsepower from tests in shallow water. For example, if the water depth is 10 percent greater than draft of vessel (depth of water/draft = 1.1), a 97 percent increase in horsepower would be

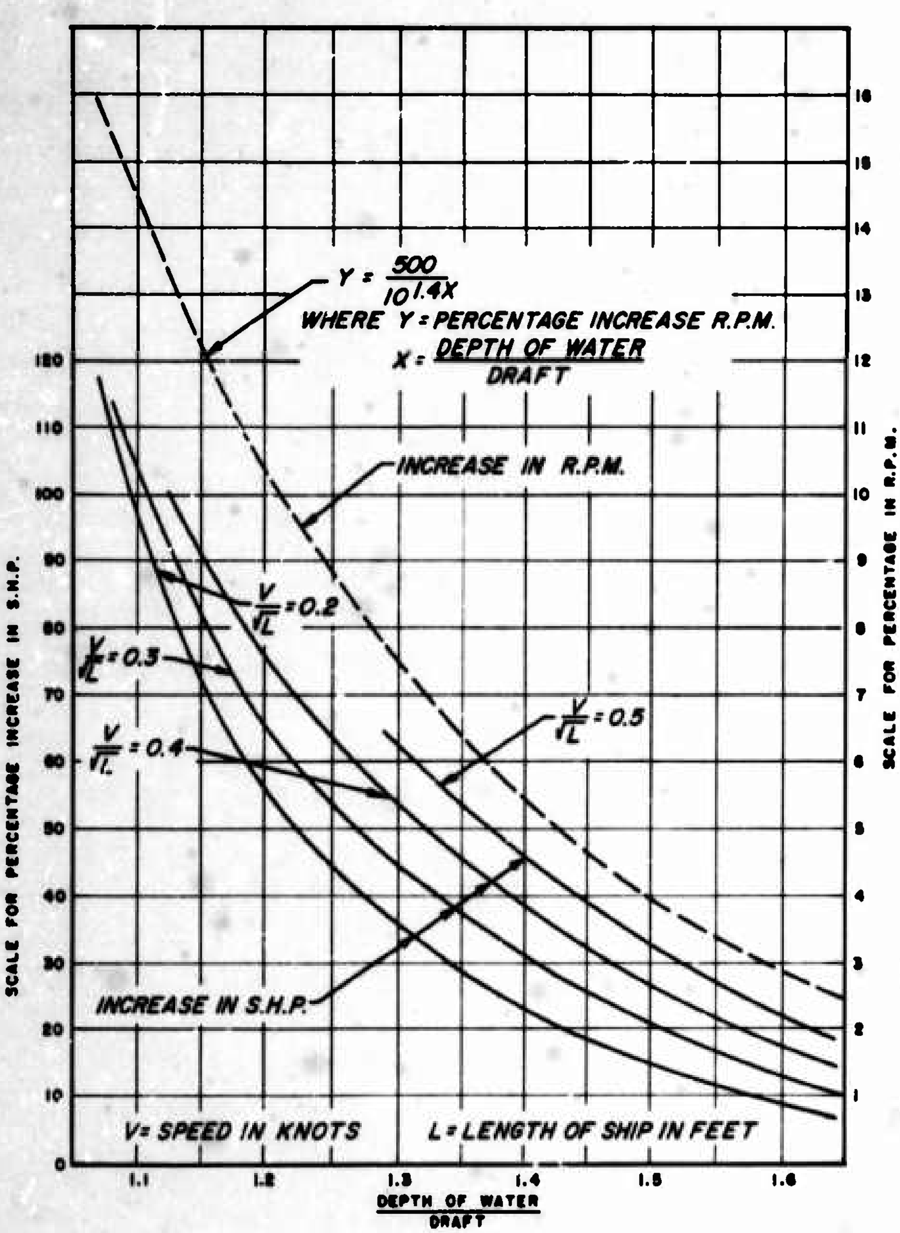


Fig. X-7. Curves of percentage increase in shaft horsepower and rpm required to maintain a speed length ratio of about 0.2; with water depth to draft ratio of 1.35, a 29 percent power increase would be required. Another source of information on this matter is reference 9.

Minimum depth under keel

X-29. The conditions that produce sinkage also produce violent flow patterns in shoal water or canals which:

a. Affect ship steering and maneuverability. No precise determination of the effect of shallow water on steering is available, but it is generally recognized that a vessel becomes hard to handle and requires large rudder angles unless speed is considerably reduced in shoal water. This will be discussed under problems of yaw and minimum waterway area.

b. Produce bed-load movements with resulting displacement of material. A vessel may readily displace a foot or two of material and leave it piled up in the way of the next passing vessel.

X-30. A nominal clearance of at least 2 ft under the keel of a vessel in motion is needed to (a) avoid damage to ships propellers from sunken timbers and debris, (b) reduce displacement of bottom material, and (c) avoid fouling of pump and condensers by bottom material. There is a great difference between touching the soft fluff which lies on the channel bottom of many estuaries and striking rock bottom or grounding on hard sand. The clearance under the keel should be increased to at least 3 ft if the bottom is rock.

Estimate of design depth

X-31. A typical calculation of minimum depth of channel for a large vessel traversing a shallow freshwater restricted channel at 8 knots follows:

30,000-DWT Ship, 650 ft in Length, Beam 86 ft

Saltwater loaded draft	35.0 ft
Added draft due to fresh water	1.0 ft
Drag (trim down at stern)	1.6 ft
Sinkage or squat at 8 knots (assume channel depth = 42 ft, and channel width = 200 ft with 1 to 1 side slopes)	1.8 ft
	39.4 ft
Minimum bottom clearance	2.0 ft
Required channel depth	41.4 ft

This result is sufficiently close to the depth assumed (in order to compute the squat by means of fig. X-1) so that it is unnecessary to try another assumed depth; if 41.5 ft is assumed as correct, it will be found that the squat computed then will be approximately the same as that for 42 ft, and the end result will again work out to be about 41.4 ft.

X-32. This depth applies only if the design vessel is not likely to pass another large vessel (not necessarily as large as the design vessel). In most cases, this is likely to happen, and it is therefore necessary to increase the depth to provide for the additional squat that will be experienced by the design vessel. This could amount to 1.8-ft additional depth in the example above, bringing the total required depth to 43.2 ft.

X-33. Before the depth is finally determined, the required depth computed as above should be examined to determine whether it is the most economic, considering the power required to propel a vessel to its destination within the waterway at a given speed as compared to that required for the same speed in a waterway of a greater depth. If the design vessel discussed in paragraph X-31 is operated at the design speed of 8 knots in water 43.2 ft deep, the power requirement, from fig. X-7, is about 75 percent greater than that required to move the vessel at 8 knots in the open ocean over great depths. If the channel depth is increased to 45 ft, the power required to move the vessel in the channel at 8 knots is about 63 percent greater than that required in great depths, and at 50 ft, the power required drops to about 38 percent greater than that necessary in great depths.

Channel Width

X-34. Table X-1 gives the beam and other characteristics of typical ships as the basis for selection of the design ship, as discussed in paragraph X-8. Channel width is determined from the beam and steering characteristics of the design vessel, from a consideration of the traffic density, and from the characteristics of other vessels encountered in the channel, as well as currents, wave conditions, winds, bends, and general alignment.

X-35. General practice is for the width of the channel to be measured at the design depth, or bottom of the side slopes. Traffic studies should be made to determine whether the design vessel is likely to meet and pass a similar vessel or smaller vessels and small boats. The design criteria would be established from economic studies as outlined in paragraph X-6.

X-36. When a vessel enters a restricted channel, not only is sea room sacrificed, but movements and controllability are affected to the extent that a vessel will frequently be moved from the course it is expected to follow. The major problem is of ship control, and this is most pronounced when ships pass each other, although the width of a channel also affects squat. In a restricted waterway, the violent flow patterns that produce strong "reverse flow" currents, high value of squat, severe scour, and shoaling action also make steering difficult. The design width should be sufficient to ensure adequate control of the ships that must use the waterway under expected conditions of ship speed, currents, and traffic. A minimum width of about five times the beam of the largest vessel is usually found necessary for two-way traffic. It is to be recognized that the maneuverability of a vessel is affected by both the width and the depth of the waterway; thus, width and depth are to be considered jointly in selecting the navigation channel dimensions.

Vessel operation

X-37. Among the factors to be considered in selection of the channel width for safe navigation are: one-way traffic or two-way traffic, overtaking and passing of large vessels or spacing between transiting ships, use of pilots' and other operating rules. Maximum vessel speed through the water is an important economic and safety consideration, along with necessary reduction in speed when passing another vessel and operating with limited visibility in fog or rain.

X-38. The handling characteristics of the using vessels is another factor to be considered. Twin-screw, single-rudder types, typical of many tankers and bulk carriers, are likely to have poor handling qualities as compared with the excellent handling qualities of most Naval vessels which have two rudders of large area located directly in the slip stream of the propellers. Maneuverability at low speed in restricted waters tends to be more difficult as the size of the ship increases.

X-39. Vessel control in a canal is likely to become difficult at high speed, particularly with a fair current when the speed over the bottom would be very great, as there is insufficient time and room for proper maneuvering, and the steering qualities are reduced in effectiveness due to suction and squat. On the other hand, low speed creates problems as a ship needs at least 5-knot water speed for proper rudder action. For very large vessels operating at low speed in critical sections of a waterway, it may be necessary to use special measures such as: (a) momentary increase of engine speed to facilitate control in a tight situation, (b) for twin-screw vessels, differential engine speeds, (c) assistance by towboat, (d) one-way traffic in critical reaches, and (e) anchoring or mooring in special areas provided when conditions become too unfavorable for safe navigation. In an extreme situation a large Naval vessel was observed to drop anchors and reverse all engines to make an emergency stop when a fleet of small sailboats obstructed the channel; this illustrates the hazard of present-day numbers of small boats.

X-40. Width requirements may be increased by other conditions:

a. Crosscurrents are critical for extremely large vessels, where speed and sea room are limited.

b. Strong winds on the beam or quarter are an important factor on partly loaded ships, or very large vessels, such as aircraft carriers traveling at low speed. Winds having velocities of 50 to 80 mph may cause a vessel to side slip 10 to 15 deg from course (yaw, or crabbing, angle).

c. Waves have to be very great for inland waters to affect the control of large ships. However, rough water and tide rips are important considerations for the smaller vessels and recreational boats, because control may be difficult and uncertain.

Effect of restricted channel width

X-41. A ship following the center line of a channel of limited width and depth will require frequent movement to correct for eddy action and small variations from her course, but the average, or equilibrium, rudder angle will be near zero. However, as soon as the vessel deviates from the center line or operates in an off-center portion of the channel, "bank suction" creates a powerful side force and yawing movement. Rudder angle to maintain equilibrium will increase with the distance from the center line, and the tendency to shear may become so great when the ship is close to the bank or an obstruction that it cannot be overcome by use of the rudder, and as a result the ship will strike one of the banks.

X-42. If we look more closely at the hydraulic phenomena generated by a ship under way on a course close to one bank of a restricted channel, we find that an asymmetrical flow distribution develops on the opposite sides of the ship which causes different water levels and unbalanced lateral forces to act upon the ship. The water level between the bow and the near bank will build up above its normal level and tend to force the bow away from the near bank, thus turning the ship towards the center of the channel. As the water flows aft to fill the void left by the ship, the current generated by the ship (see paragraph X-11) in the confined area between the hull and the near bank is greatly increased. This results in a drop in water level and pressure (Bernoulli's theorem), and the stern of the vessel is forced towards the near bank. The adverse effects on the ship will be increased by (a) poor steering characteristics of the vessel, (b) the nearness of the bank or obstruction and narrowing of overall channel width, and (c) shallow depth under keel. The behavior of the vessel is likely to follow one of several patterns:

a. Rudder angle is adjusted to counteract the force turning the bow away from the bank and to maintain the center line of the ship (not the course) parallel to the bank. If this is maintained, the vessel will be moved bodily into the near bank. Once a moving ship is very close to one bank or an obstruction, it is difficult to move away without striking the stern. A contributing factor is the rudder action which swings the stern in the opposite direction before the bow moves along the new course (like driving an automobile backwards; i.e. the front end first moves opposite to the turning direction).

b. If the vessel is allowed to make a slight angle away from the bank, the side force will be counterbalanced by giving the rudder just sufficient angle for equilibrium so as to maintain a course parallel to the bank or slowly work towards the center of the channel. Under these conditions the yaw angle, or side slip (crabbing), of the vessel will develop a force opposing the bank suction.

c. If the rudder angle or control is insufficient, the bow will be forced away from the bank, the stern sucked in towards the bank, and the vessel will be turned across the channel. As the bow approaches the far bank, the differential pressures (high pressure outside, low pressure inside) will tend to turn the bow into the bank. Once a vessel takes a shear from its normal course in a narrow channel, it is liable to strike either bank. Examples of the rudder angle required for equilibrium for a large Naval vessel operated at various speeds at various distances from the near prism line of channels of several depths and widths are given in fig. X-8. The "G" referred to in these graphs is the center line of the ship. The "rudder angle for equilibrium" is defined as that angle that causes the course of the ship (not the longitudinal center line of the ship) to be parallel with the side of the channel. In general terms, these data show that the equilibrium rudder angle for a given channel increases with speed, but the rate of increase is somewhat lower in the wider and deeper channels.

Two-way traffic in a restricted channel

X-43. The effect of a passing vessel is to form an obstruction that accentuates the effects of the narrow width and shallow depth. As two ships approach and meet near the center line, the high pressure area between the

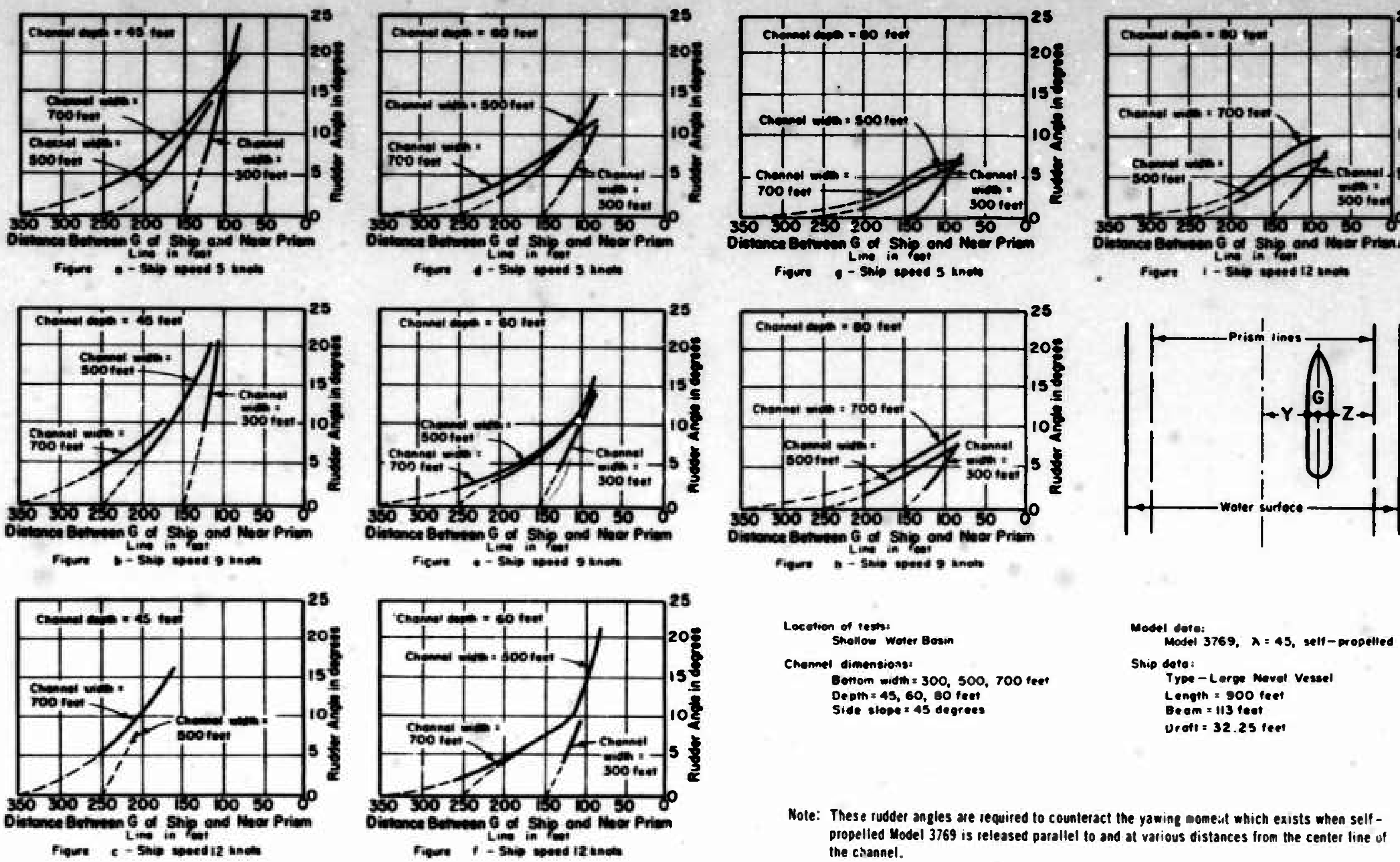


Fig. X-8. Rudder angle for equilibrium as a function of the distance between G of ship and near prism line of channel bows will tend to cause the ships to yaw away from each other; this action is followed by strong suction, which tends to draw the ships together. As the maximum safe speed for two-way traffic is likely to be 30 to 40 percent less than for one-way traffic in the same channel, a prudent navigator will reduce speed when passing another vessel even if waterway rules do not require that this action be taken.

Estimate of required channel width

X-44. The width of the channel is measured at the bottom of the slope, i.e. at the design depth, which is either the required depth for safe navigation of the design vessel or the economic depth. Some of the factors that must be given consideration in determining the proper width of the channel are: whether the design vessel must pass a similar vessel, or a smaller vessel or vessels; the controllability of the vessel; the normal speeds of the vessel relative to the channel bottom; current velocities and directions; wave action or wind that will cause the vessel to yaw; the depth of water under the keels of the vessel; whether the channel occupies the entire waterway, as in a canal, or is located in a wide waterway many times the width of the channel; and the characteristics of the banks of the channel, i.e. whether they are rocky or composed of soft sediments.

X-45. There is no formula or equation that takes account of all of the preceding diverse factors. However, there are data that furnish guidance for the design. Most of these are the result of the investigations made during the study of the proposed sea level Panama Canal.[5,10,11] After careful analysis of the data developed during a considerable program of experimentation in the existing prototype Panama Canal and with models in model channels, the engineers in charge of the investigation formulated procedures for the determination of the widths of a proposed sea level Panama Canal that are useful for making a first approximation of a channel to accommodate any selection of vessels in any given waterway.

Width of maneuvering lane

X-46. The first step is the determination of the width of the maneuvering lane. This is defined as that portion of the channel within which the ship may "maneuver," i.e. deviate from a straight line, without encroaching on the safe bank clearance or without approaching another ship so closely that dangerous interference between ships will occur. The experimental data reveal the values shown in table X-4.

Table X-4

Maximum Width of Vessel Lane, Model 3769, Beam 113 ft
(Large Naval Vessel)

	Width of Maneuvering Lane in Channels As Indicated			
	45- by 300-ft Channel	60- by 300-ft Channel	45- by 500-ft Channel	60- by 500-ft Channel
Average for 5 knots, ft	179	167	152	159
Average for 6 knots, ft	---	165	---	---
Average for 7 knots, ft	174	---	150	166
Average for 8 knots, ft	---	168	---	---
Average for 9 knots, ft	193	175	149	159
Average for 10 knots, ft	---	187	---	---
Average of all observations, ft	180	174	151	161
Ratio of average of all channel width observations to vessel beam, %	159	154	134	142
Maximum lane observed, ft	282	210	205	215
Ratio of maximum channel width to vessel beam, %	250	186	181	190

It is seen that the width of the lane is only slightly affected by ship speed in the range of 5 to 10 knots, but that it tends to become somewhat less in the wider and deeper channels. The Panama Canal and the Taylor Model Basin engineers concluded that the Naval vessel handled well, and that the width of the maneuvering lane should be taken as 160 percent of the beam. In a similar manner, but without benefit of as much data, they concluded that the width of the maneuvering lanes for a large tanker (beam 100 ft) and a Liberty Ship (beam 57 ft) should be 180 percent of the beam on the basis that these ships do not handle as well as the large Naval ship. They also concluded that the width of the lane should be 160 percent of the beam of a vessel of "very good" controllability, 180 percent of the beam of a vessel with "good" controllability, and 200 percent of the beam of a vessel with poor controllability.

X-47. The tests discussed previously were made under laboratory conditions with currents that tended to run parallel with the channel and without winds or waves that would cause the vessel to deviate to a greater extent. However, observations in the existing Gaillard Cut of Panama Canal on the Aircraft Carrier LEYTE produced results that were said to be similar, and no reason is seen to doubt the applicability of the conclusions of the Panama Canal engineers to prototype considerations. The question arises as to whether, disregarding currents at an angle to the channel and winds, the width of the lane would be materially different in a channel occupying a small portion of the width of a waterway from that required in a restricted channel. Examination of the detailed data from which the data contained in table X-4 were derived shows that there is little correlation between the width of the lane and the location of that lane relative to the center line of the channel, except when the ship is close to the edge of the channel. From this, it appears that the width of the lane is to a considerable extent based on the peculiarities of the ship and the diligence and skill of the helmsman. These factors would exist in channels in open water as well as in restricted channels; in fact, it is likely that there would be less diligence employed by the helmsman while traversing a wide channel in open waters that do not have crosscurrents or transverse winds than would be employed in a restricted channel.

X-48. In waters where there is a current at an angle to the channel, or where wave action and strong winds tend to cause the vessel to yaw, the maneuvering lane width should be increased beyond the values discussed previously. Data on the yawing characteristics of the design vessel and other vessels to be accommodated in the channel under design may be obtained from the owners or masters of the vessels. A 10-deg yaw of vessels in channels traversing large bodies, or in bar channels at entrances, would not be unusual. A vessel 800 ft in length and 100 ft in beam yawing 10 deg would swing back and forth through a lane 240 ft wide. If the lane width under calm conditions with no transverse current should be 180 ft for this vessel, there is no guidance for a decision as to whether to make the lane width for conditions producing the 10-deg yaw 180 ft + 240 ft = 420 ft, or some lesser value.

X-49. Summarizing, it appears that the width of the maneuvering lane may be as little as 160 percent of the beam of the design vessel where it is known that the design vessel has very good controllability and will be operated by men of skill and diligence. This value appears to be applicable for channels in wide waterways as well as restricted channels where there are no currents at an angle to the channel, or winds or waves that will cause vessel yaw. In places where these yawing forces exist, the width of the maneuvering lane should be that required to accommodate the oscillations of the vessel as it yaws back and forth, which is determined by the length of the vessel and the angle of yaw that may be expected or the 160 percent of the beam figure, whichever is greater. It is considered that these widths are minimal, and should be used only in cases where it is clear that the vessel has very good controllability and will be handled expertly. When consideration is given to the disaster and economic loss that occur when great ships collide, or the damages suffered when they go aground, it is likely that the more conservative lane width of 180 percent of the beam of the vessel will be employed for reaches where there are no yawing forces, or perhaps even 200 percent of the beam of the design vessel in cases where that vessel is known to have poor controllability. For reaches where yawing forces are likely to be experienced, the lane width premised on a percentage of the vessel beam might be increased for the yaw as a result of this kind of judgment.

Width of ship clearance lane

X-50. In cases where the channel is required to accommodate two-way traffic involving the larger vessels, a ship clearance lane must be provided between the two maneuvering lanes. It is taken to be the distance between the inner boundaries of the maneuvering lanes, as the ships could be in this position during the passing operation. Both vessels are subject to bank suction and the interaction between the vessels. Model tests made during the Panama Canal investigation revealed that interaction between the passing vessels created no appreciable hazard when the distance between them was equal to the beam of the larger ship, and after consultation with the Panama Canal pilots this criterion was adopted. It is likely that the width of the clearance lane could be reduced in channels in a waterway many times the width of the channel, but only for reaches that are well buoyed and not subject to strong currents or strong yawing forces. A minimal width of clearance lane amounting to 80 percent of the beam of the larger vessel might be provided for such circumstances.

Bank clearance

X-51. Fig. X-8 has already been discussed briefly, but it should be examined in greater detail. It will be observed that the distance between the "G" of the ship and the near edge of the channel is a function of the equilibrium rudder angle, the width and depth of the channel, and the speed of the vessel. A sampling of the data presented in fig. X-8 may be replotted, as in fig. X-9, to show more clearly how the width of the bank clearance lane varies with depth and width for two rudder angles and speeds. The width of the bank clearance lane in these graphs is expressed as a ratio of the width in feet of the lane to the beam of the ship in feet. These graphs show clearly that a wider lane is required for a 5-deg equilibrium rudder angle than for a 10-deg angle; for a channel 45 ft deep and 700 ft wide, the lane width must be about 100 percent of the beam of the vessel for a 10-deg angle and 150 percent for a 5-deg angle. They also show that a wider lane is required for a given rudder angle at a higher speed than at a lower

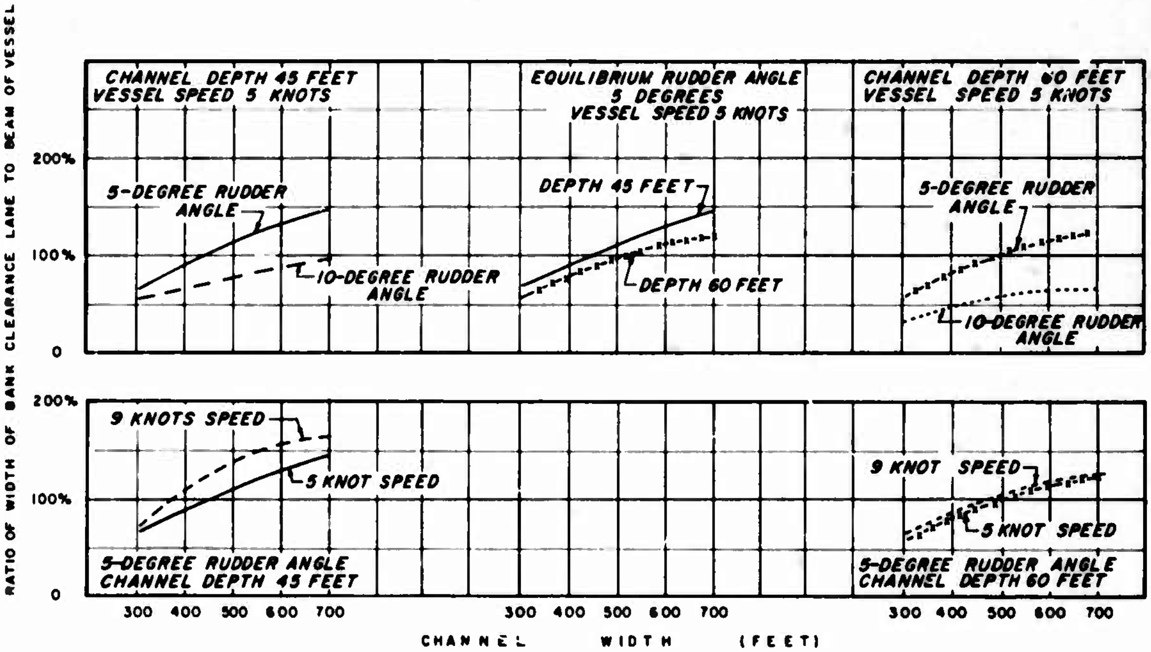


Fig. X-9. Restricted-channel bank clearance lane width for large naval vessel (beam = 113 ft) speed; for a channel 45 ft deep and 700 ft wide and a 5-deg rudder angle, the bank clearance lane should have a width of 150 percent of the beam of the vessel while it is moving at 5 knots, while the width of the lane should be increased to about 165 percent if the vessel is to move at 9 knots safely. The graphs show that increased depth permits use of a narrower bank clearance lane for a given rudder angle, a given channel width, and a given speed. Finally, the graphs show that for a given rudder angle and a given speed, the bank clearance lane must be increased in width as the total channel width is increased. Presumably, some maximum width of bank clearance lane is reached in each case, beyond which further increases in total channel width do not require further increase in bank clearance width. At first consideration, it may be surprising that a greater bank clearance width is required for a 700-ft channel, for example, as compared to a 400-ft channel. In the wide channel, reaction from the far bank is much lower than the reaction from the near bank, and it is necessary to increase the width of the bank clearance lane to reduce the difference between these two forces to a value that can be offset by the indicated equilibrium rudder angle. In the narrow channel, the intensities of the reaction from the near bank do not differ from that of the far bank as greatly.

X-52. The experimentation performed during the course of the Panama Canal investigations revealed that the required bank clearance is also a function of the particular characteristics of the vessel involved. Tests were performed on a large tanker with a beam of 100 ft and a Liberty ship with a beam of about 57 ft. The results showed that the bank clearance lane should be somewhat wider for the large tanker than for the large Naval vessel of the David Taylor model tests (figs. X-8 and X-9). Direct comparison is not possible, as the tests for the tanker were run at somewhat different values than were used in the tests on the Naval vessel. However, by interpolation it was ascertained that ratios of the lane width to beam for the tanker must be about 70 percent greater than the ratio of the lane width to the beam of the large Naval vessel. Thus, it would be computed that the width of the bank clearance lane required for a tanker operating at 9 knots in a 45- by 500-ft channel with a 5-deg rudder angle would be 240 percent of the 100-ft beam, or 240 ft, as compared to 140 percent of the 113-ft beam of the Naval vessel, or 160 ft. A rather meager amount of experimentation with the Liberty ship model resulted in the finding that this ship apparently does not require as large a lane, relatively, as either the Naval vessel or the tanker. For example, at 5 knots and a 5-deg rudder angle in a 45- by 500-ft channel, the width of the lane is found to be only 33 percent of the beam, as compared to 110 percent for the Naval vessel.

X-53. The Taylor Model Basin and the Panama Canal engineers concluded that the bank clearance should be based on a rudder angle of 5 deg, which they considered to be "very conservative but selected to provide an excess of safety to cover conditions that exist in an actual waterway that cannot be reproduced in the model." In accepting their views and evaluating the results summarized in paragraph X-52, it appears unwise to accept a bank clearance lane width of less than 60 percent of the beam of the vessel and unduly conservative to provide more than 150 percent of the beam of the design vessel without additional evidence to support lower or higher values.

X-54. All of the experimentation was performed in restricted channels, and the data do not provide much evidence as to the differences that would exist between the results for such channels and channels of similar dimensions in wide waterways. It appears reasonable, however, to assume that there would be little difference between fully restricted channels and partially restricted similar channels, i.e. channels in wide but shallow waterways where the width of the waterway is many times the width of the channel but the depth beyond channel limits is much less than that within the channel. It also appears reasonable to assume that there would be a greater difference in cases where the depths beyond the limits of the channel are only a little shallower than those in the channel. However, there are other factors that require consideration. These include the existence of strong currents, currents at an angle to the channel, winds and waves at angles to the channel, whether the material beyond the limits of the channel is rocky or is hard sands or gravels, the difficulty of determining the exact limits of the channel, and whether the depths within the channel close to the edges usually are somewhat less than those closer to the center line, due to shoaling. These factors cause masters of vessels to be reluctant to maneuver their vessels close to the edge of the channel, regardless of whether the channel is fully restricted, partially restricted, or is in a wide and fairly deep waterway. Where they exist, the width of the bank clearance lane should rarely be less than 150 percent of the beam of the design vessel. Where they do not exist, the width could be as little as 60 percent of the beam of the design vessel if it is known to handle well that close to the edge of a channel. The application of the criteria discussed above is illustrated in table X-5.

Widening at bends

X-55. Curves are more difficult to navigate than straight channels because of: (a) the reduction of clear sight distance and reduced effectiveness of navigation aids, such as ranges; (b) with constantly changing heading

Table X-5

Computation of Channel Width

Maneuvering Lanes

Criteria	Application			
	Vessel	Beam ft	Lane Width, ft	
			160% of Beam	180% of Beam
Required width is 160% to 200% of the beam of the design vessel, depending on its controllability.	T-2 tanker Super tanker Large Naval vessel Victory ship	75 100 113 57	--- --- 181 ---	316 180 --- 102

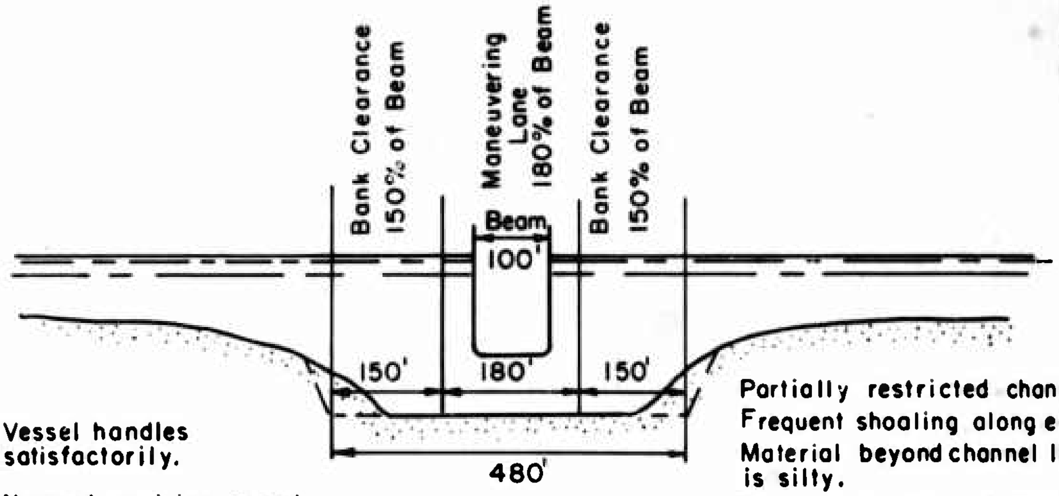
Ship Clearance Lane

Criteria	Application
100% of the beam of the design vessel, or 100 ft, whichever is greater, when waterway is restricted or subject to strong yawing forces. In channels that are well buoyed and not subject to strong yawing forces, a width equal to 80% of the beam of the design vessel.	Case 1. Restricted waterway; super tanker likely to pass Liberty ship. Clearance lane width should be 100 ft (minimal for this condition). Case 2. Channel is in a wide waterway and is well buoyed; large Naval vessels are likely to meet Victory ship. Clearance lane width should be 80% of 113 ft = 90 ft.

Bank Clearance

Criteria	Application
150% of the beam of the design vessel in channels where there are strong yawing forces, or where the edges of the channel are subject to recurring shoaling, or where the material beyond the channel limits is rocky or hard sands or gravels. Minimum width is 60% of the beam of the design vessel where these conditions do not exist and the vessel is known to handle well that close to the edge of the channel.	Case 1. Design vessel will not pass another large vessel. Channel subject to frequent shoaling at edges. Design vessel is a super tanker. Provide 150% of 100 ft each side of maneuvering lane. Case 2. Design vessel (a super tanker) likely to meet a Victory ship. Channel is in a rock cut. Provide bank clearance lane on one side of 150% of 100 ft, and on the other, 150% of 57 ft. If the design vessel is likely to pass another design vessel, make each lane 150 ft wide.

Note: Examples of the combination of the designs of the various lanes into a channel are given in fig. X-10.



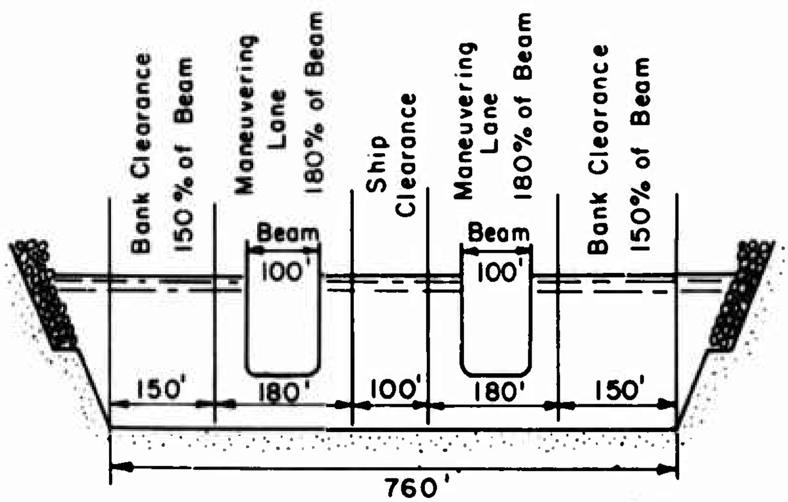
Vessel handles satisfactorily.

Normal cruising speed in this channel is 8 knots, relative to bottom.

Partially restricted channel.
Frequent shoaling along edges.
Material beyond channel limits is silty.
Strength of current 2 knots.

Current flows parallel with channel.

Moderate to strong winds at an angle to channel occur frequently.



Vessel handles satisfactorily.

Normal cruising speed in this channel is 5 knots, relative to bottom.

Fully restricted channel.
Revetted banks.
Strength of current 4 knots.

Current flows parallel with channel.

Strong winds rare.

Fig. X-10. Elements of channel width

of ship, navigation is difficult in fog or strong wind; (c) the forces causing control problems in restricted waterways are more pronounced on curves, and the ship itself forms a tangent or secant to the curve and is positioned off center; (d) poor flow conditions and eddies are likely to occur which subject the ship to crosscurrents and bank suction forces; (e) changing channel area or section creates changes in strength of current; and (f) the rudder and centrifugal force on a vessel making a sharp turn tends to displace the stern to a path well outside the path of the bow (see fig. X-11).

X-56. Although widening of the channel at turns may cause undesirable flow conditions that tend to make steering more difficult, negotiation of an unwidened turn having a deflection angle greater than some certain value for a given vessel would be infeasible for routine operations in the waterway; extraordinary skill and great caution would be required to keep the ship in the channel. As pointed out before, the path of a vessel making a turn is wider than is its path in a straight reach of the channel, and it is generally believed that the greater the deflection angle, the greater will be the width of the path. It is also considered that the speed of the vessel relative to water, the length of the ship, its beam, and its steering characteristics are factors in determining the width of the path in a turn. Similarly, the presence of another ship in the turn and the existence of extraneous yawing forces such as currents, winds, and waves at angles to the changing course of the ship as the turn is made result in a wider path. Accordingly, it is customary to widen the channel in turns, but unfortunately there is not much guidance available to determine the amount of widening that is desirable. The most extensive body of information that may be employed in the formulation of criteria was assembled during the course of the Panama Canal investigation, but the engineers in charge of this study found that it was not entirely adequate for their purpose, although it was the basis of their somewhat arbitrarily determined widenings; it was their intention to resume the investigation in the event that the sea-level project was authorized for construction. Although these data have not been verified by prototype tests, they are considered to be of value and are presented in table X-6. It will be seen that only three different turns and two vessels were studied, and that one series of tests was made at a scale of 1:86 while a less extensive series was made at a scale of 1:45. The Panama Canal engineers concluded that the 1:45 tests were of greater value than the 1:86 tests, but evidently time did not permit repeating all of the 1:86 tests at the larger model scale.

X-57. It is found that there are no trends in the variation of width of path for a given vessel speed relative to water for different current velocities

Fig. X-11. Ship making sharp turn showing stern displaced to path for outside the path of the bow

Table X-6

Width of Path of Vessels in Turns

Channel Current, Velocity, and Direction	Width of Path, ft											
	Vessel Speed, 5.0 knots				Vessel Speed, 7.5 knots				Vessel Speed, 10.0 knots			
	40-deg Turn (Widened), 108- by 860-ft Vessel	26-deg Turn (Widened), 108- by 860-ft Vessel	26-deg Turn (Un-widened), 108- by 860-ft Vessel	26-deg Turn (Un-widened), 113- by 900-ft Vessel	40-deg Turn (Widened), 108- by 860-ft Vessel	26-deg Turn (Widened), 108- by 860-ft Vessel	26-deg Turn (Un-widened), 198- by 860-ft Vessel	26-deg Turn (Un-widened), 113- by 900-ft Vessel	40-deg Turn (Widened), 108- by 860-ft Vessel	26-deg Turn (Widened), 108- by 860-ft Vessel	26-deg Turn (Un-widened), 108- by 860-ft Vessel	26-deg Turn (Un-widened), 113- by 900-ft Vessel
No current	360	475	290	277	466	450	369	305	723	378	309	209
3-knot head	763	639	238	245	643	411	263	245	401	402	342	231
5-knot head	---	---	---	---	649	430	297	257	568	417	378	231
3-knot fair	512	---	229	217	622	---	235	183	486	---	253	177
5-knot fair	474	557	257	246	544	430	284	236	512	399	304	209

Note: The 40-deg widened turn consists of a curve between tangents having a deflection angle of 40 deg. The channel is widened to 1088 ft; it is 60 ft deep.

The 26-deg widened turn consists of a curve between tangents having a deflection angle of 26 deg. The channel is widened to 872 ft; it is 60 ft deep.

The 26-deg unwidened turn consists of a curve between tangents having a deflection angle of 26 deg. The channel is 560 ft wide and 60 ft deep, as in the straight reaches.

All tests of the 26-deg unwidened turn with the 113- by 900-ft vessel were made at a scale of 1:45, while all other tests were made at a scale of 1:86.

and directions. It is therefore appropriate to average the results for each vessel speed and kind of turn tested. The results are tabulated below:

Table X-7

Width of Path of Vessels in Turns

Ship Speed knots	Width of Path in Feet				
	40-deg Widened Turn 108- by 860-ft Vessel	26-deg Widened Turn 108- by 860-ft Vessel	26-deg Un-widened Turn 108- by 860-ft Vessel	26-deg Un-widened Turn 113- by 900-ft Vessel	Avg
5.0	527	557	257	246	397
7.5	544	430	284	236	373
10.0	512	399	304	209	356
Average	531	449	282	230	374
Ratio*	4.90	4.15	2.61	2.04	

* The ratio indicated is determined by dividing the average width of the path by the beam of the vessel.

These average figures do not confirm the belief that the width of path increases with an increase in vessel speed, as may have been expected; in fact, three of the four series of numbers show that the width becomes less with increase of speed, but it is difficult to rationalize this and additional test material might show a different result. The averages show that the width of path is indeed a function of the size of the deflection angle, as the width for a 26-deg widened turn is 449 ft while the average path width for a widened 40-deg turn is 531 ft. However, it is also seen that the width of the path in the unwidened 26-deg deflection angle turn is much less than that of the widened 26-deg turn, possibly because more care may have been exercised in maneuvering the vessel around the turn in the narrower channel, or because of the effects of the side of the channel. This may indicate that the path for a 40-deg turn would not be as much wider than that for the 26-deg turn if both turns were widened in the same amount; table X-6 shows that the 40-deg turn was widened to 1088 ft, while the 26-deg turn was widened only to 872 ft. Finally, it is seen that the vessel with a beam of 113 ft made a path that was narrower than the vessel having a beam of 108 ft, probably because it had superior steering characteristics, although the fact that the tests were made at different scale values may have been a factor.

X-58. After considering various methods for applying the above values in the design of the widening of a turn, it appears that the most reasonable procedure would be to premise it on the three factors that the Panama Canal engineers established for the design of the maneuvering lanes in straight reaches, namely, 160 percent of the beam of a vessel that has very good controllability, 180 percent of the beam of a vessel that has good controllability, and 200 percent of the beam of a vessel that has poor controllability. It may be deduced that the vessel with the 108-ft beam in tables X-6 and X-7 should have a maneuvering lane of 200 percent of its beam when in a straight reach, based upon the figures in table X-7 and the 160 percent figure used by the Panama Canal engineers for the 113-ft vessel. This permits derivation of the following for the width of the maneuvering lane in 26- and 40-deg turns:

Table X-8

Maneuvering Lane Width in Straight Reaches and in Turns

Controllability of Vessel	Width of Maneuvering Lane As a Percent of the Vessel Beam		
	Straight Channel	Channel Turns with Deflection Angles of	
		26 deg	40 deg
Very good	160	325	385
Good	180	370	440
Poor	200	415	490

X-59. When these criteria are applied to the cases illustrated in fig. X-10, the following results are obtained:

Table X-9

Channel Widths in Straight Reaches and in Turns

Design vessel is 100- by 733-ft tanker

Channel Element	Channel Width, ft					
	For One-Way Traffic			For Two-Way Traffic		
	Straight Channel	26-deg Turn	40-deg Turn	Straight Channel	26-deg Turn	40-deg Turn
Bank clearance	150	150	150	150	150	150
Maneuvering lane	180	370	440	180	370	440
Ship clearance	---	---	---	100	100	100
Maneuvering lane	---	---	---	180	370	440
Bank clearance	150	150	150	150	150	150
Total	480	670	740	760	1140	1280
Increase in width for turn	---	190	260	---	380	520

X-60. All of the tests utilized in developing the criteria summarized in table X-8 were performed in a fully restricted channel. There is some question as to the applicability of the criteria for widening turns in channels in wide waterways, but the same reasoning given in the discussion (paragraph X-47) of the applicability of data on required maneuvering lane widths in straight reaches is applicable to turns. It appears that the width of the lane is to a considerable extent based on the peculiarities of the ship and the diligence and skill of the helmsman. As these factors exist in channels in wide waterways as well as in fully restricted channels, no reason is seen for a finding that the widening may be less in a turn in a channel located in a wide waterway; in fact, if there should be a difference, it might be that the widening should be greater than in a restricted waterway, for it is likely that the helmsman will be especially diligent in the restricted channel and less so in the channel in a wide waterway.

X-61. It may be thought that the radius of curvature of a turn is perhaps a more valid criterion as to the amount of widening required than is the size of the deflection angle. There are empirical equations in the literature that purport to establish such relations, as stated below:

$$\Delta W = 2R - (4R^2 - L^2)^{1/2}$$ (Proposed by F. V. de Carvalho - XIV PIANC Congress)

$$\Delta W = 85 - \frac{R}{100}$$ (Kiel Canal, 1926)

$$\Delta W = 4[R - (R^2 - L^2)^{1/2}]$$ (Ghent-Terneuzan Canal, 1926)

where

ΔW is the increase in width required to negotiate the curve

R is the radius of the curve at the channel center line

L is the length of the ship

Table X-10 gives the ΔW values according to the above equations for one-way transits of the 100- by 733-ft tanker considered in table X-9.

X-62. The Panama Canal engineers examined these equations and rejected them, stating that their bases could not be ascertained. It is of course reasonable to accept a premise that there is a relation between the length of the design vessel on the one hand, and the width and radius of curvature of the channel on the other hand. However, it is unrealistic to ignore the size of the deflection angle completely, as do the three equations. In order to negotiate a turn, it is of course necessary to turn the rudder and to hold it more or less

Table X-10

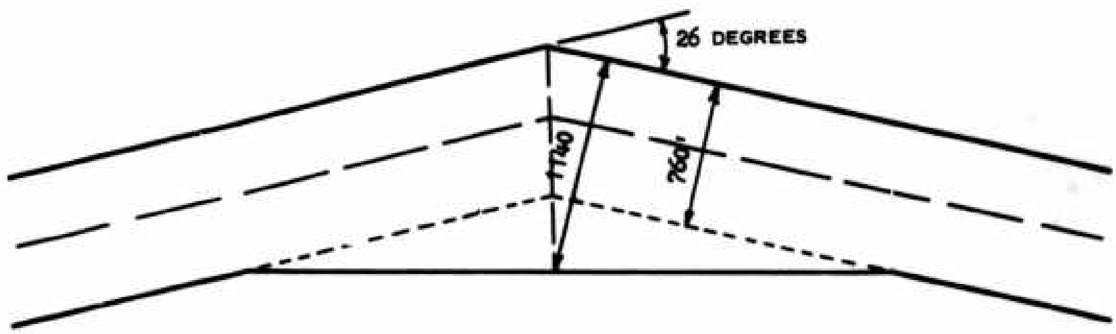
Widening of Channels at Turns, Based on Radius of Curvature

Radius of Curve at Channel Center Line, ft	Required Widening in feet According to Equation of		
	Carvalho	Kiel Canal	Ghent-Terneuzan Canal
2,500	54	60	440
5,000	28	35	216
7,500	20	10	144
10,000	14	--	108
12,500	11	--	88
15,000	9	--	72

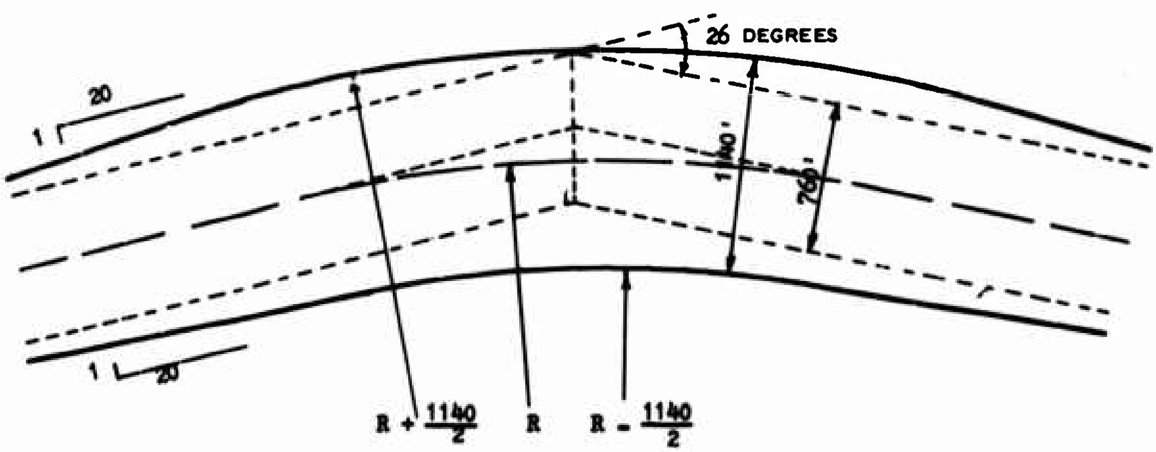
constantly at the required angle throughout the maneuver. Adjustments will be necessary as the head swings too far in one direction or the other, perhaps to an even greater extent than when the ship is on a straight course, but the latitude for adjustments to one side of the turning angle is much less than on the other. This causes steering difficulties, and it would seem likely that these would compound as the length of time that the rudder must be held off midships increases, i.e. as the length of the curve increases. It is apparent that the length of curve for a given radius of curvature increases as the size of the deflection angle increases. It therefore seems that the three equations given previously do not adequately define the required widening; a factor should be included for the length of the curve, or the deflection angle. On the other hand, it could be argued with equal validity that widening based entirely upon the deflection angle, as is the case for the criteria summarized in table X-8, is inadequate; perhaps the widening should be greater, for a given deflection angle, for smaller radii curves than for larger.

X-63. The Panama tests were performed in turns having radii of 12,500 ft. It might be well to adjust the values determined by the criteria in table X-8 by amounts similar to those tabulated under the Carvalho heading, for example; this is suggested because they are reasonably checked by the Kiel Canal values. Thus, for the 100- by 733-ft design vessel, the widening would be 190 ft for a 26-deg turn having a radius of 12,500 ft, and 190 ft + (54-11) ft = 233 ft for a 26-deg deflection angle turn having a radius of 2500 ft. Considering the meager amount of basic information available for the development of the widening criteria, this step appears to be an unjustifiable refinement.

X-64. The required widening may be introduced into the channel in a number of ways. The most common method, known as the Apex, or Cutoff, Method, consists of extending the outer prism lines until they intersect and cutting off the angle that would be formed by extension of the inner prism lines to a straight line (fig. X-12). The Panama Canal engineers, after considering the



WIDENING BY THE APEX, OR CUTOFF, METHOD



WIDENING BY THE PARALLEL BANKS METHOD

Fig. X-12. Two methods of channel widening evidence of their model experiments showing that this method of widening introduced undesirable current conditions, developed empirically the more refined widening method consisting of providing a channel of constant width throughout the curve, then gradually reducing that width to the normal width of the channel in straight reaches. This may be called the Parallel Banks Method. Fig X-12 illustrates this layout. It is immediately evident that the Parallel Banks Method would involve considerably more excavation than a turn widened by the Apex, or Cutoff, Method. The only evidence as to the benefit to be derived from the added cost pertains to fully restricted channels, where an abrupt change in cross-section geometry due to the widening at the turn would certainly cause undesirable variations in the currents. However, it is unlikely that a similar increase in width of a channel in a wide waterway would cause a similar variation of the currents in the channel. It seems reasonable to reserve the Parallel Banks Method for use in restricted channels, and that the Apex, or Cutoff, Method be used for widening channels at turns in wide waterways.

Widening of long tangents

X-65. Although the straight channel is normally considered the ideal condition, it occasionally introduces problems. From the navigation viewpoint, a ship or small boat sailing on a long tangent is likely to drift from the straight and narrow channel as a result of crosswinds, waves and currents, or small errors in helmsmanship and navigation. Also, a straight channel may deviate somewhat from its dredged location as strong tidal currents cause shoaling and erosion. While these conditions may not be serious in fair weather, when an accurate fix of the vessel's position can readily be obtained and the outline of the deepwater channel can often be observed from color of the water and the pattern of waves and current, consideration of fog and bad weather may require widening a long tangent, even if the channel is well marked by navigation aids.

Widening of entrance channels

X-66. Entrance channels are frequently subject to strong and variable tidal currents, rough seas, breaking waves, wind, fog, and other navigation difficulties. Bar channels and entrances partly protected by jetties and training works will require special studies of tidal currents, shoaling, and littoral drift to determine optimum relations between entrance width, cross-section alignment, and exposure. In such areas control is likely to be difficult for both large ships and small boats so that channel width must be judiciously selected.

Design considerations for determining
the alignment of the channel

X-67. The overriding requirement for channel alignment is that all vessels expected to use the channel be able to navigate it with reasonable safety under adverse conditions of tide, current, and wave and wind action in the channel. Consequently, the minimum permissible radius of curvature or maximum deflection angle at bends is governed by the turning characteristics of the least maneuverable ship. Turning ability is dependent upon rudder response. Adequate steerageway and depth under keel are necessary to assure design rudder response. Furthermore, external forces upon the ship can reduce control, greatly increase the turning radius required, and lengthen the time and distance necessary to complete the maneuver. Therefore, appropriate safety factors to compensate for such adverse conditions must be provided for in the determination of minimum acceptable radius of curvature in the channel. Widening at bends and the effects of interacting forces between the ships and channel banks have already been discussed.

X-68. Casual observation would be that a straight channel has the advantage of being the shortest route and easiest to maintain. However, experience has been that the forces of nature seldom cooperate wholeheartedly and frequently are so powerful that man-made straight channels prove difficult and expensive to maintain. The regimen of a waterway in its natural state is the resultant of all the forces of nature having acted upon it over the centuries. Incorporation of the natural alignment to a practical degree can reduce or concentrate shoaling problems with resulting minimum maintenance costs. The channel through coastal lowlands and across the shoreline to deep water may be much less stable than upstream reaches. Here the violent vagaries of the sea, high tides, and storm waves may in a few hours undo the cumulative effects of years of moderate influences.

X-69. In selecting the alignment for a new or improved channel, the engineer must balance the anticipated benefits from straightening against those of following the natural alignment or a compromise design between the two. In addition to the basic comparison of initial construction costs, the costs of maintenance for considered alignments must be given detailed attention, for they can well determine the most economical channel. Cross-section changes and realignment invite counteractions by nature. The combined influences from tidal action, changed current patterns, Coriolis effect, freshwater flow, bank and bottom disturbances, and littoral drift at the entrance will affect the stability and maintenance requirements of the channel.

X-70. Critical locations for a ship navigating a channel are at the ocean entrance, bridges, and approaches to locks, barriers, and control structures.

Here straight approaches, long enough for vessels to become properly aligned, are a necessity.

X-71. In the entrance channel, vessels may encounter strong ebb, flood and cross currents, transverse winds, severe wave action, and rough water intensified by the relative shallowness of the channel. Entrance channels are particularly hazardous for both ships and small craft because of these outside forces and the limited area for maneuvering. A straight entrance channel parallel to the resultant of these forces (with due allowance for the direction of storm waves) is generally the safest for navigation, but type of material and shoaling influences frequently require adjusted alignment to obtain economic maintenance.

Channels for Small Vessels

X-72. Although the procedures described in the foregoing sections of this chapter were developed for the design of channels for large vessels, they may be applied to the design of channels for small vessels, where appropriate. Under most circumstances, a nominal squat of 1 ft may be assumed for normal or safe speeds, and a 2-ft clearance under the keel while under way will be added. Where wave action is a factor, due consideration will be given to additional depths commensurate with the expected pitching of the boat. The width of channels for small boats in fully restricted waterways will be based on the width design considerations contained herein, with 50 percent added to the width computed by these methods when the currents are unusually strong, say in excess of 5 knots. The width of channel in wide waterways ordinarily will not be in excess of 100 ft, even if the currents are in excess of 5 knots; the exception would be in cases where the traffic density is unusually high. Channels for small craft should be widened at turns, using design procedures similar to those described in preceding paragraphs.

Special Considerations

X-73. In the design of navigation channels, many factors peculiar to each location or plan will require study and special consideration. Factors to be considered include:

- a. Safety, efficiency, and time of transit.
- b. Transit rules, regulations, and restrictions that will be required.
- c. Daily vessel traffic, including size, volume, and characteristics of ships and small boats.

d. Length of restricted channel reaches.

e. Currents and waves under normal operating conditions and during storm conditions and their regulation by control structures.

f. Climatic conditions, including fog, wind, and rain that may slow traffic or make layovers necessary.

g. Operational dependability during normal or wartime operation, considering accidents and special operating conditions.

h. Maintenance requirements and alternative navigation channels or routes needed during maintenance and construction periods.

i. Construction cost and time.

j. Increased ship resistance and power required in a restricted channel.

k. Need for mooring areas, emergency tie-up stations, and turnaround areas.

l. Clearance for berthed vessels outside the channel line.

m. Adaptability of design to future improvements to meet the needs of increased number and size of vessels, tributary channels, and new harbor and docking areas.

Ease of maintenance

X-74. The efficiency of the design with respect to shoaling and maintenance is an essential consideration. The design of a channel for ease of maintenance must, of course, go hand-in-hand with design in the interest of safe and economic navigation. It is frequently necessary to strike a compromise to keep maintenance dredging within reasonable bounds, where the optimum channel conflicts with ease of maintenance. In consideration of such problems, it should be recognized that shoaling frequently increases with increase of channel width and depth. Also extensions of channels to upper reaches of estuaries may shoal more rapidly than downstream reaches. Channels across the thalweg of the waterway and extensions into tributary streams, coves, and small harbors usually require high maintenance. Increased channel depths sometimes have a profound effect on the distribution of shoaling as well as the rate of shoaling, as discussed in Chapter V; shoals may occur in dock areas remote from suitable spoil disposal areas, with resultant great increases in dredging costs.

Channel currents

X-75. Current velocities not only affect shoaling, but also are a major consideration in safety of navigation and channel design. Crosscurrents and nonuniform flow are particularly hazardous to large vessels, where the bow may encounter currents quite different from the stern 1000 ft away where the steering and power are supplied. Increased channel width is necessary where currents are strong to compensate for the increased difficulty of control and ship handling. It will frequently be desirable to increase the waterway area of constricted sections, if this will reduce the current velocity or provide more uniform current conditions. Particularly in entrance channels, a strong ebb current, resulting from tidal currents and freshwater river flow, running against prevailing wind and waves is likely to generate violent "tide-rip" conditions or standing waves unfavorable to navigation. Such entrances are avoided by navigators in bad weather. Model studies are usually needed to obtain a satisfactory solution of such problems as the state of knowledge does not permit exact solutions based on hydraulic analyses or experience for the wide variety of conditions which are encountered.

Economic analysis

X-76. Good engineering will, of course, require economic studies including comparisons of the estimated benefits and costs to achieve the most economical alignment and channel designs and to formulate the best overall project giving full consideration to economical ship operation, safety, water quality, pollution, recreation, fish and wildlife, and other intangible factors. Adaptability of the design to future improvements and navigation growth is a major consideration.

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